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Update of Intersection/Interchange Guidelines for Rural Expressways in Nebraska

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16. Abstract

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This project updated and expanded NDOT's guidance using contemporary traffic operational and safety data, combined with robust microsimulation and safety performance modeling. Traffic operations data were collected from TWSC, RCUT, roundabout, and signalized intersections on rural Nebraska expressways, as well as from diamond interchanges with stop-controlled and signalized ramps and a diverging diamond interchange. These facilities were modelled tin a microsimulation environment calibrated to Nebraska driving behavior, generating more than 11,500 scenarios covering a wide range of major road, minor road, and ramp volumes, turning combinations, and time-of-day conditions. The simulation results were used to develop statistical models for estimation of operational delays and predictions of the most suitable intersection/interchange alternatives.

Safety analysis incorporated crash data from TWSC intersections along rural expressways to develop a negative binomial model for estimation of crash frequencies across different traffic scenarios. The safety performances of alternative at-grade facilities were estimated using appropriate crash modification factors. This study used the FHWA "Interchange Comparison Safety Tool" to estimate crash frequency at conventional diamond and diverging diamond interchanges.

Furthermore, the research team compiled construction and retrofit cost information from national sources and Nebraska-specific projects for a comprehensive benefit-cost analysis.

The resulting guidelines integrated benefit-cost outcomes with operational and safety analyses, engineering judgment, and national best practices. These data-driven guidelines provide NDOT with a modern, consistent, and defensible framework for evaluating intersection upgrades, selecting interchange types, and assessing grade-separation needs on Nebraska's rural expressways.

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Disclaimer

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Abstract

Nebraska Department of Transportation (NDOT) staff frequently decide on changes to existing two-way stop-controlled (TWSC) intersections on rural expressway to alternative facilities, such as a restricted crossing U-turn (RCUT), a roundabout, a signalized intersection, or an interchange. As Nebraska makes progress toward completion of its expressway system, updated and defensible decision-making is essential for conversion of TWSC highway intersections. NDOT's existing guidelines, developed in the 1990s, no longer reflect current operations, safety performance, or modern intersection designs.

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Chapter 1 Introduction

1.1 Background

Nebraska Department of Transportation (NDOT) staff routinely decide whether an existing two-way stop-controlled expressway intersection should be converted to another facility type, such as a signalized intersection, roundabout, or interchange. These decisions will remain necessary as NDOT progresses toward completing the statewide expressway system. The primary guidance currently available for such decisions was contained in "*Interchange vs. At Grade Intersection on Rural Expressways*" (Bonneson and McCoy, 1992). The report evaluated conversions from two-way stop-controlled intersections to either a signalized intersection or a full diamond interchange with stop-controlled ramps. Its recommendations were based on the best information available at the time, including the 1985 Highway Capacity Manual (HCM, 1985), research from the 1960s through the early 1990s (e.g., Tanner, 1962; Fitzpatrick et al., 1993), accident studies by Council and Paniati (1990), and construction cost estimates from FHWA (1988) and AASHTO (1977). This guidance is now outdated and no longer reflects current traffic and safety characteristics, design practices, and modern intersection and interchange options.

This research addressed these limitations by updating the decision-making framework using recent cost data, contemporary analytical methods, and Nebraska-specific operational and safety information. The updated methodology focused on three major components: (1) assessment of operational efficiency and delay costs using state of the art microsimulation calibrated and validated for Nebraska conditions and latest HCM method; (2) estimation of safety performance and crash costs using recent crash data and contemporary crash modification factors; and (3) compilation of construction and retrofit costs based on recent market conditions and project experience. In addition to updating the methods used in the 1992 guidance, this research incorporated several newer facility types not previously considered, including roundabouts, Restricted Crossing U-turns, and Diverging Diamond Interchanges. NDOT has already implemented several RCUTs and one DDI, with additional installations underway and planned in the future, making the inclusion of these options essential for relevant and practical guidance.

This project was particularly timely given NDOT's planned expansion of expressway corridors, including US 77 between Wahoo and Fremont, US 81 between York and Columbus, and US 275 between Norfolk and West Point. These projects require clear, current, and comprehensive guidance on whether and how to convert two-way stop-controlled intersections. The existing 1992 guidelines do not address the full range of available alternatives and do not reflect current crash and delay costs or modern traffic analysis tools.

Overall, this research fulfilled three critical needs: (1) updating NDOT's existing intersection and interchange decision guidelines; (2) expanding the guidance to include additional modern facility types; and (3) applying the latest analytical tools and most current data relevant to Nebraska conditions. These updates provide NDOT with a complete and contemporary basis for making intersection/interchange conversion decisions along rural expressways.

1.2 Research Objectives and Tasks

The project had the following objectives.

1. Assess the operational and safety performance and associated costs of different alternative design options available for reconfiguring rural TWSC expressway

intersections based on the current state-of-the-art method adjusted to Nebraska conditions.

2. Assess the construction/retrofit costs of different options that are available for intersections/interchanges.
3. Develop rural expressway intersection reconfiguration guidelines based on benefit-cost analysis of the alternative design options.

This research focused on the following types of intersection and interchange options: TWSC, roundabout, Restricted Crossing U-turn, signalized intersections, Diamond Interchange with stop-controlled ramp, Diamond Interchange with signal-controlled ramp, and Diverging Diamond Interchange.

This project was conducted with the following seven major tasks.

- Task 1: Review of literature
- Task 2: Data collection
- Task 3: Operational analysis and associated cost estimations
- Task 4: Safety analysis and associated cost estimations
- Task 5: Construction/Retrofit costs estimations
- Task 6: Benefit-cost analysis and establishment of guidelines
- Task 7: Preparation of final report

1.3 Report Outline

This report is organized as follows. Chapter 2 reviews the related literature. Chapters 3 and 4 model and analyze unsignalized intersections and interchanges, respectively. Chapter 5 discusses the safety performance of unsignalized intersections and interchanges. Chapter 6 presents the operational and safety evaluation of signalized intersections. Chapter 7 addresses the analysis of grade-separation alternatives in terms of operation and safety. Chapter 8 focuses on the conduction of the benefit-cost analysis, which formed the basis for the final guidelines presented in Chapter 9. Chapter 10 concludes the report with a summary, conclusions, and recommendations for future research.

Chapter 2 Literature Review

This chapter presents a literature review of comparative operational and safety aspects of different intersections and interchanges related to this project.

2.1 Operations and Safety of Different At-Grade Intersections

2.1.1 TWSC and RCUT

Minor road stop-controlled intersections are the most common form of traffic control along rural highways and expressways. At a four-legged intersection, stop controls are provided at two minor approaches, making it a two-way stop-controlled (TWSC) intersection. Traffic on the minor approaches to the TWSC intersection must stop and yield to traffic on the major road, which operates under free-flow conditions (HCM, 2022). Drivers on the minor road must wait for an acceptable gap before entering or crossing the high-speed major road traffic stream. The TWSC intersection is widely used because it operates efficiently under typical rural conditions, where minor road traffic volumes are relatively low. At these locations, the TWSC intersection provides a simple, low-cost, and operationally effective means of managing right-of-way. When minor road volumes remain low to moderate, operational delays are minimal, and overall the intersection operates at a high level of service (Kyte et al., 1991). Because of this simplicity in traffic control and ability to handle typical rural demand, the TWSC intersection continues to be the predominant intersection type across Nebraska's rural roadway network.

Although the TWSC intersection performs well operationally when minor road traffic is low, it is also associated with a higher potential for severe crashes. Drivers on the minor approach must judge gaps in high-speed traffic on the major road, increasing the likelihood of right-angle and turning collisions. These crash types tend to be more severe, making safety a key concern even when operational performance appears acceptable for the TWSC intersection.

The Highway Safety Manual (HSM, 2010) provides crash prediction models for TWSC on two-lane and four-lane highways. Apart from HSM, several other research projects examined the safety performance of minor road stop-controlled intersections. Stapleton et al. (2019) developed safety performance functions for rural, low-volume, minor road stop-controlled intersections using safety data from 83 counties in Michigan. This study limited the major road volumes to a range of 400 to 2,000 vehicles per day, and data were collected from 2,023 intersections over the time period 2011 to 2015. Besides providing safety performance functions, this study found that HSM models overpredicted crashes at intersections for major road traffic volumes within 1,200 and 2,000 vehicles per hour. In another study, Sun et al. (2024) developed safety performance functions for TWSC intersections on rural two-lane highways in Louisiana. Using a negative binomial model with 5,126 TWSC intersections, the authors found that AADT, curve radius, and skewness angle significantly affected crash frequency. The results showed that Louisiana-specific safety performance functions outperformed HSM-calibrated outcomes, suggesting that the original HSM model overpredicted crashes in Louisiana.

The literature suggests that the safety performance of stop-controlled intersections is of interest to traffic agencies and researchers, as they are the most prevalent type of intersection in rural settings and often pose substantial safety concerns. Therefore, transportation agencies often retrofit stop-controlled intersections into alternative types of intersections to improve traffic operations and safety. Reconstruction may involve converting TWSC intersections to unsignalized designs such as roundabouts and restricted crossing U-turns (RCUTs).

RCUTs, also known as Superstreets or J-Turn intersections (Hughes et al., 2010), gained widespread acceptance, with more than 250 installations documented across the United States

(Hughes et al., 2010; Hummer et al., 2014; Sangster and Adams, 2019; Ulak et al., 2020). Table 2.1 lists several example states with known deployments (Ozguven et al., 2019).

Table 2.1 RCUT implementations in states (modified from Ozguven et al., 2019)

States	**Number of RCUTs**	**Signalized**	**Unsignalized**
Alabama	11	6	5
Georgia	23	0	23
Illinois	1	0	1
Indiana	3	0	3
Louisiana	5	0	5
Maryland	14	1	13
Michigan	3	2	1
Minnesota	12	0	12
Mississippi	8	0	8
Missouri	19	0	19
Nebraska	3	0	3
North Carolina	118	25	93
Ohio	3	3	0
South Carolina	3	0	3
Tennessee	4	0	4
Texas	5	5	0
Wisconsin	8	0	8
Total	*243*	*42*	*201*

TWSC is effective at managing traffic in low-volume scenarios but may be compromised at higher traffic volumes, leading to increased difficulty and confusion over right-of-way, especially for minor road left-turning vehicles (Early et al., 2016). RCUT modifies TWSC operations by eliminating direct left-turn and through movements from the minor road; instead, drivers turn right and complete their movement via a U-turn crossover on the major road (Hummer et al., 2014) as shown in Figure 2.1. RCUT usually results in crash reduction and enhanced operational efficiency for high-volume major roads and light-to-moderate minor roads, but the advantage may diminish for moderate-to-high minor roads (Al-Omari and Abdel-Aty, 2021).

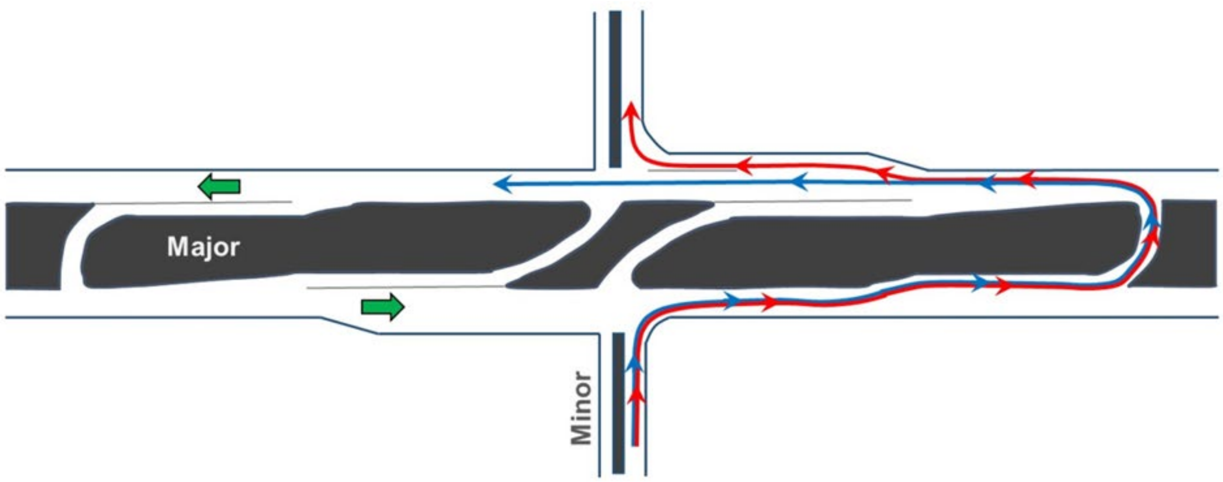


Figure 2.1 Schematic diagram of RCUT intersection showing movements from minor approach (Ultak et al., 2020)

Individual studies on the RCUT are readily available in the literature. However, this project is focused on the comparative analysis of intersection types. To date, research comparing TWSC and RCUT comprehensively remains limited.

While RCUT offers significant operational benefits, comparisons in the literature between RCUT and TWSC intersections have yielded mixed results, depending on traffic composition, left-turn proportions, roadway geometry, and acceleration lane provision. As mentioned, RCUT improves minor left-turn and through movement efficiency by rerouting them via U-turns to avoid conflicts with major roads; however, this arrangement adds delay. Thus, for many combinations of traffic volumes and turn proportions, whether RCUT outperforms TWSC remains complex and requires comprehensive evaluation.

Hughes et al. (2010) reported that signalized RCUT reduced network travel time by 40% when minor approach volume ratios were 0.2 or less, compared to intersection volumes, and performed well under heavy major left-turn volumes and moderate minor volumes. Edara et al. (2010), evaluating unsignalized RCUT with 27,000 annual average daily traffic (AADT) and TWSC with 17,000 AADT, within the same level of service (LOS), observed five seconds less delay at the minor approach but 60 seconds more in travel time for RCUT. Note that RCUT in this study restricted major left-turn movements, unlike typical configurations where they are permitted (Edara et al., 2010). Also, this reduction in wait time at the minor approach occurred due to the inclusion of acceleration lanes. At locations without such lanes, wait times for right-turn movements would likely increase due to the need to find an acceptable gap in major road traffic. In another study, Haley et al. (2010) reported in their simulation study that RCUT exhibited lower travel-time standard deviation than equivalent conventional intersections, indicating that even when travel times increased on minor approaches at a few sites, the experienced travel time became more consistent and reliable.

RCUT performance is sensitive to the balance between major and minor road volumes. Naghawi and Idewu (2014) measured the effect of RCUT implementation on traffic performance measures using CORSIM. The greatest delay and queue length differences occurred when 30% of minor road left-turns occurred with 15% left-turns from the major approach. Hummer et al. (2012) showed that RCUTs performed best when major road traffic was at least twice the volume of minor roads.

Regarding lane-changing behavior and U-turn design of RCUT, Xu et al. (2017) applied a linear regression approach to identify the traffic volume at which an unsignalized RCUT begun to lose its effectiveness. It was based on the likelihood of vehicles changing lanes from the minor road to the downstream U-turn bay. Using a U-turn offset of 1,500 feet, the threshold was found to fall between approximately 2,500 and 3,000 vehicles per hour (vph). On the other hand, Kim et al. (2007), using the FHWA's Surrogate Safety Assessment Model (SSAM), found one-lane RCUT designs could experience about 80% fewer conflicts than comparable conventional intersections. However, the benefits were reduced for two-lane U-turn configurations, where increased lane-changing activity raised the potential for rear-end conflicts. Edara et al. (2016) researched the design guidelines for RCUTs, focusing on safety features and U-turn locations. Key findings included that most major road-side swipe and rear-end crashes occurred while vehicles were merging with traffic or changing lanes to enter a U-turn. These crashes were attributed to higher speed differentials between merging major road vehicles and driver inattention. However, these types of crashes decreased with increased spacing between the minor road and the U-turn, and consequently, RCUTs with a spacing of 1,500 ft or greater experienced the lowest crash rates.

The safety literature consistently finds substantial crash reductions at RCUT installations. Conflict points at RCUT are distributed across multiple locations rather than concentrated at a single intersection, as in TWSC configurations, allowing drivers to manage fewer movements at once. The reduction in conflict exposure contributes to improved safety performance (Lu et al., 2005). Overall, RCUT contains 18 conflict points compared to 32 at conventional intersections, demonstrating a substantial potential of safety advantage associated with its geometric design (Bared, 2009).

Edara et al. (2015) compared RCUT facilities with TWSC intersections and analyzed crash histories from five locations in Missouri. It was found that total crashes declined by 54.4% after RCUT implementation. Crashes related to disabling injury and minor injury reduced by 91.6% and 67.9%, respectively. Using an empirical Bayes approach, this research attributed 28% to 34.2% of the crash reduction to the RCUT design. Additionally, for minor road turning vehicles, time-to-collision conflict measures increased 4-fold in RCUT compared to the TWSC intersection, indicating higher safety. Research conducted by Evans et al. (2012) for FHWA supported many of the findings reported by Edara et al. (2015). They observed reductions in property-damage-only crashes, fatal crashes, and overall injury crashes. Specifically, fatal crashes declined by 70%, and injury crashes dropped by 42% over the two three-year periods examined (Evans et al., 2012).

Inman et al. (2013) examined the safety performance of RCUT on four-lane divided highways and found that they reduced total crashes by about 44%, including a 9% reduction in injury and fatal crashes. RCUT was especially effective in lowering angle collisions while adding only minimal travel time. Consistent with other studies, the authors also noted that acceleration lanes could help reduce delays and enhance overall traffic flow. Hummer et al. (2010) reported substantial reductions in total, angle, right-turn, and left-turn crashes at unsignalized RCUTs, along with notable decreases in fatal and injury collisions.

Ott et al. (2011) evaluated 13 unsignalized RCUTs in North Carolina and recommended a CMF of 0.46. Their findings showed that RCUTs are highly effective at reducing crashes on rural divided four-lane arterials and are best suited for locations where low-volume two-lane roads meet high-volume divided highways. They also noted that the comparison group method was appropriate for evaluating RCUT safety, as regression-to-the-mean effects were minimal. However, RCUTs may not be ideal where minor street left turn and through demands are high.

In summary, RCUT was found to be superior to the TWSC intersection in terms of safety and operations across various traffic conditions.

2.1.2 TWSC, Roundabout and Signalized Intersections

The roundabout is a circular intersection where entering vehicles yield to circulating traffic, enabling continuous flow from all approaches (NCHRP Report 1043, 2023). Many traffic agencies consider roundabouts at rural high-speed locations (Brewer et al., 2023; Zhao et al., 2015). A roundabout's capability to increase safety is well documented along with its ability to accommodate varying traffic demands (NCHRP Report 1043, 2023; Brewer et al., 2023). There are more than 9,000 roundabouts across the United States, making it one of the most implemented unsignalized intersection options. Understandably, traffic agencies often consider converting a TWSC to a roundabout.

Flannery et al. (1998) recommended a single-lane roundabout for major road volume within 150-850 vph, compared to a TWSC intersection. On the other hand, Eustace (2000) found that within 250–650 vph and left-turns up to 60%, roundabouts had higher mean delays but lower variability than TWSC intersections. In other words, travelers experienced high travel time reliability while navigating roundabouts. Luttrell et al. (2000) reported similar findings when comparing roundabouts to TWSCs in Manhattan, Kansas.

Notably, Yang et al. (2017) evaluated the operations, safety, and environmental impacts of converting TWSCs to yield, all-way stop-controlled, and roundabout intersections by applying AADT of 16,000 and 6,000 for major and minor roads, respectively. They found that, for AADT between 7,100 and 5,600, the net present value became positive, favoring roundabouts. Note that this calibrated TWSC used the VISSIM microsimulation traffic modeler with mean observed travel times but could not calibrate such a model for roundabouts due to a lack of field data (Yang et al., 2017). In contrast, Brewer et al. (2023) simulated single-lane roundabouts in VISSIM using field data and TWSC intersections, without field validation, in high-speed rural two-lane two-way locations. They showed that at 5,000 average daily traffic (ADT), delays for both TWSCs and roundabouts were under 20 seconds per vehicle (sec/veh), and at 10,000 ADT, roundabouts stayed below 50 sec/veh, while TWSCs exceeded 150 sec/veh. This study demonstrated the efficacy of roundabouts at high traffic demands compared to TWSC intersections.

Besides TWSC, traffic agencies often consider converting a signalized intersection to a roundabout as well. In terms of operations, roundabouts are expected to produce fewer operational delays than signalized intersections at similar traffic volumes (Rodegerdts et al., 2010). For MUTCD peak-hour signal warrant threshold volumes, roundabouts outperformed signalized intersections, producing lower average intersection delays.

While roundabout provides operational advantages compared to TWSC and signalized intersections, its safety benefits are even more pronounced. Isebrands (2009) conducted a crash safety analysis at 17 high-speed rural intersections that were converted to roundabouts. They found substantial safety benefits, with roundabouts declining the average injury crash frequency by 84% and the average injury crash rate by 89%, while the angle crashes were reduced by 86%, and fatal crashes were eliminated entirely.

In another study, Isebrands et al. (2014) investigated how the type of intersection—roundabout or TWSC—affects the approaching speed at rural high-speed intersections. It was found that the mean speed 100 feet from the yield line of the roundabout was 2.5 miles per hour (mph) lower than the mean speed 100 feet from the stop bar at TWSC intersection approaches. This observation demonstrated that a roundabout could force all drivers to reduce their speed at the intersection. This speed drop contributes to reducing injury crashes.

Rodegerdts et al. (2007) conducted a before-and-after study comparing traditional stop- or signal-controlled intersections with roundabouts and found that roundabout conversions decreased both total and injury crash rates across urban, suburban, and rural settings by 35% for total and 76% for injury.

Isebrands (2012) evaluated rural roundabouts located on high-speed approaches and conducted both a before–after analysis using a negative binomial regression model and a before–after empirical Bayes assessment. The two methods produced consistent findings. The negative binomial model indicated a 63% reduction in total crashes and an 88% reduction in injury crashes across 19 high-speed rural roundabouts. The EB analysis showed similar improvements, with total crashes reduced by 67% and injury crashes by 87%. In another study conducted in Wisconsin, Qin et al. (2011) evaluated 24 roundabouts using empirical Bayes before-after approach. Across all sites, total crashes declined by 9.2%**,** while fatal and injury crashes were reduced by 52%**.** Among the eight rural roundabouts in the study, total crashes declined by 45% and fatal and injury crashes by 56%. For the 11 roundabouts with posted speed limits of 45 mph or higher, reductions of 34% in total crashes and 49% in fatal and injury crashes were observed.

Khattak et al. (2014) analyzed Kansas data on single-lane roundabouts and found that converting TWSC intersections to modern roundabouts led to reductions across nearly all crash types. Total crashes dropped by 58%, with fatal crashes eliminated entirely and non-fatal injury crashes reduced by 77%. Property-damage-only crashes decreased by 36% overall, although two of the four study sites experienced increases in this crash category following the conversion. For the nine rural roundabouts examined in this study, all converted from TWSC intersections, total crashes decreased by 72% and injury crashes declined by 87%. Among the 24 suburban roundabouts converted from either signalized or TWSC intersections, total crashes were reduced by approximately 42% and injury crashes by 68%.

Converting the roundabout from a TWSC and signalized intersection showed clear safety benefits. On the other hand, converting TWSC to a signalized intersection yielded mixed safety findings, which were discussed in greater detail in the main body of this report. This literature review briefly discusses the safety implications of signalized intersections.

HSM (2010) provided safety performance functions for signalized intersections located at four-lane rural highways. Recently, Tobic et al. (2022) proposed safety performance functions for three-legged and four-legged signalized intersections on high-speed urban and suburban arterials, based on crash data from California, Illinois, and Minnesota.

In terms of safety comparisons between facility types, HSM (2010) reported that, for rural intersections, converting from a stop-controlled intersection to signal control was associated with an overall crash modification factor (CMF) of about 0.56 across all crash types and severities combined. Right-angle crashes, which are often the most severe at TWSC intersections, showed a much larger reduction, with a CMF of about 0.23, while left-turn crashes have a CMF of about 0.40. At the same time, rear-end crashes increased, with a CMF of about 1.58. These values were reported to apply to rural intersections where the major road AADT is roughly 3,200 to 30,000 and the minor road AADT is about 100 to 10,300 vehicles per day. Additionally, Srinivasan et al. (2014) evaluated signal installations with and without left-turn phasing at rural intersections and reported similar patterns.

On the contrary, recent studies from Iowa indicated an increase in crashes at signalized high-speed intersections (Abukhalil et al., 2025). Additionally, Qiao et al. (2019) reported higher predicted crash rates at signalized intersections than at TWSC intersections when evaluating the cost-effectiveness of these two facilities in terms of operations and safety.

In summary, roundabouts outperformed TWSC and signalized intersections in terms of safety and, for many traffic scenarios, also provided superior operational performance. On the other hand, the safety implications of converting a TWSC to a signalized intersection are contested.

2.1.3 Roundabout and RCUT

Comparison between RCUTs and roundabouts is uncommon, with only a few studies (Sangster and Adams, 2019; Fernandes and Coelho, 2022) covering the topic. Sangster and Adams (2019) reported that single-lane roundabouts had lower delays and 10% higher benefits than RCUT, though their findings were based on site-specific traffic and simulated RCUT performance without field validation.

Another non-US study (Fernandes and Coelho, 2022) comparing the conversion of a three-legged single-lane roundabout into a turbo-roundabout or RCUT found that both alternatives performed better, with the turbo-roundabout having a slight edge.

2.2 Research Gap of At-Grade Intersections

While several studies developed safety implications with a proper statistical modeling approach for unsignalized intersections, predictive model-oriented studies comparing operational studies are rare as most studies rely on graphical presentations to compare results. On the contrary, Appiah (2021) used logistic regression to develop signal warrants for RCUTs by comparing operational performance between alternatives. Such predictive modeling is essential for informed intersection selection decisions, particularly across diverse traffic conditions. Therefore, this project aimed to develop data-driven predictive models for operational outcomes to support intersection selection decisions. Note that while safety outcomes often drive intersection selection decisions, the site-specific nature of these outcomes may limit their broad applicability. In contrast, operational performance tends to be more geographically consistent and often guides decision-making when safety studies are not feasible or produce inconclusive results (Haque et al., 2024).

The literature revealed several gaps in research for at-grade intersections:

1. Most comparative studies focus solely on performance metrics without incorporating cost implications.
2. Analyses often use different methods, ranging from macroscopic approaches to microscopic simulations and seldom apply a uniform or field-calibrated framework across all intersection types.
3. Simulation-based studies typically calibrated models using mean values rather than full distributions of field performance, despite evidence that distribution-based calibration is crucial.
4. Findings are difficult to generalize from the literature due to variations in roadway classification (e.g., two-lane vs. four-lane highways), inconsistent lane configurations, and heterogeneity within RCUT designs (e.g., signalized vs. unsignalized, left-turn policies).
5. No comprehensive and field-calibrated study directly compares TWSCs, roundabouts, and RCUTs under a unified methodology and provides guidelines for traffic agencies.

2.3 Operations and Safety of Different Grade-Separated Interchanges

Transportation agencies often reconstruct existing interchanges to improve traffic operations and safety. The most common interchange type in rural and suburban areas of the United States is a diamond interchange (DI), often referred to in the literature as a conventional diamond interchange (CDI). A DI connects a freeway/expressway with a cross-street using two one-way ramp terminals that form a pair of closely spaced intersections (i.e., ramp terminals). The expressway traffic enters and exits via the ramps that meet the cross street at these ramp terminals. A DI can be equipped with stop-controlled ramps (DIstop) or signalized ramp terminals (DIsig). Traffic agencies may retrofit existing DIstop to DIsig to meet operational and safety needs. However, over the past two decades, the diverging diamond interchange (DDI), also known as the double crossover diamond interchange, has gained widespread attention as a viable option, with over 200 applications in the United States (Hughes et al., 2010; Cunningham et al., 2021; DDI, 2025). Figure 2.2 shows statewide implementation of DDI at different levels (DDI, 2025).

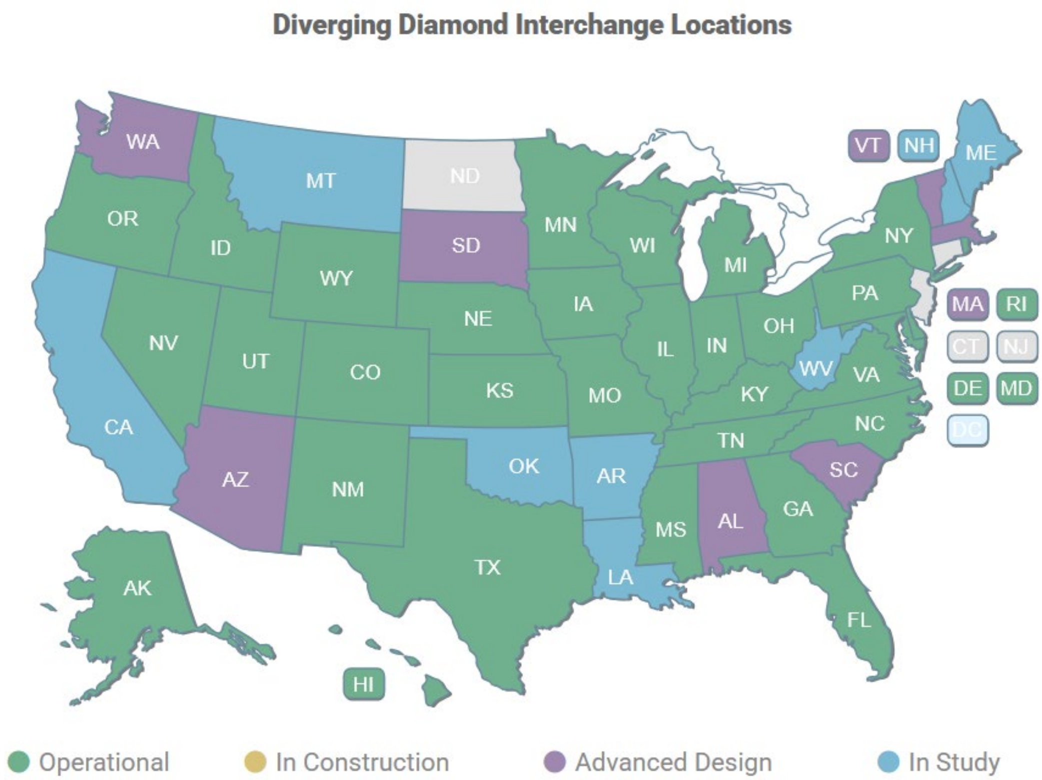


Figure 2.2 DDI applications across states as of 2025 (DDI, 2025)

The primary difference between a DDI and a DI is the design of directional crossovers on both sides of the interchange. DDI design eliminates the need for left-turning vehicles to cross the paths of approaching vehicles. DDI shifts the cross-street traffic to the left side of the street between the signalized crossover intersections. This way, vehicles on the cross-street approach making a left turn onto the on-ramp, or vehicles from the off-ramp making a left turn to cross the street, do not conflict with vehicles approaching from opposing directions (Cunningham et al., 2021). Figure 2.3 demonstrates traffic movements at DDI (Wisconsin DOT DDI, 2025).

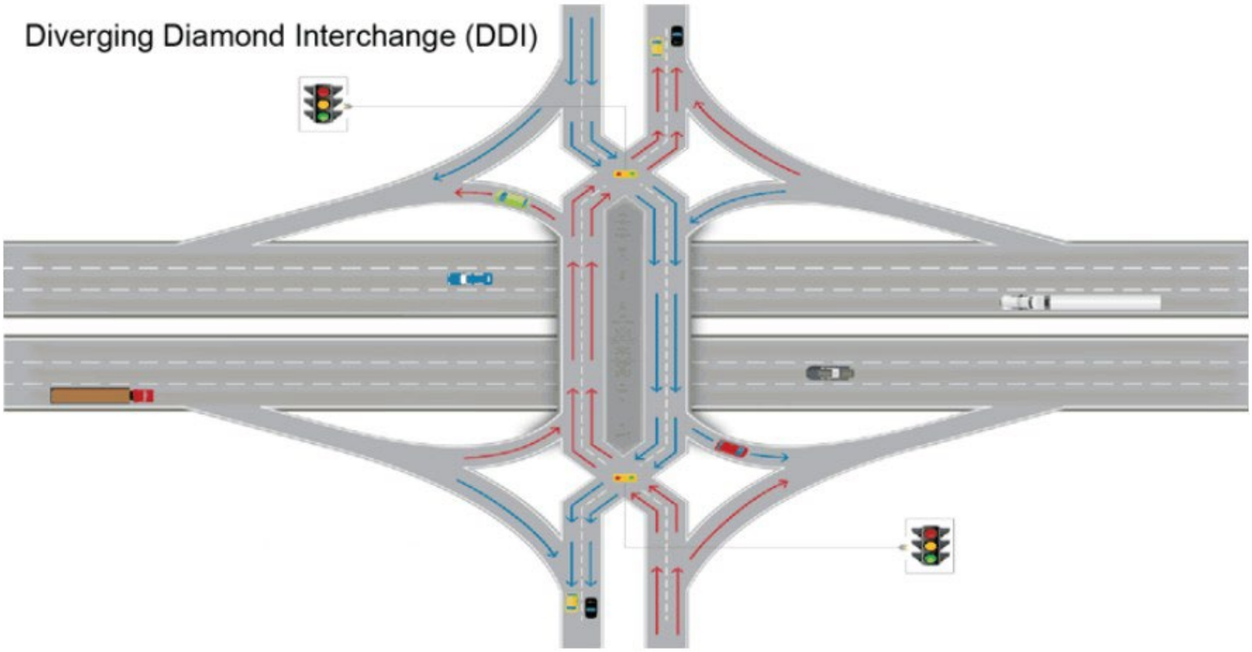


Figure 2.3 Traffic movements at DDI (Wisconsin DOT DDI, 2025)

As mentioned earlier, DIstop connects a major highway with a minor cross-street or arterial via a pair of one-way stop-controlled ramps in each direction. Therefore, vehicles exiting the ramps are required to stop at the cross-street. DIstop effectively operates under low to moderate traffic volume conditions, but under heavier traffic its operation is accompanied with increased delays and greater safety concerns. As a result, traffic agencies may convert DIstop to DIsig, where ramp terminals are managed with traffic signals to improve efficiency and safely accommodate higher traffic volumes. However, even DIsig has operational limitations, particularly during high traffic with heavy left-turning movements (Cunningham et al., 2021). DDI, as an alternative, reduces conflict points, simplifies signal phasing to just two phases (compared to six to eight in a DIsig), and can often be implemented within the existing footprint, avoiding costly overpass bridge widening (Hughes et al., 2010; Cunningham et al., 2021).

The concept of DDI was introduced by Chlewicki in 2003, and the first operational DDI in the United States opened in Missouri in 2009 (Chlewicki, 2003; Molan et al., 2017). Before 2009, a notable study by Bared et al. (2005) discussed the design and performance of DDI compared to similar designs. Simulation studies were conducted using the VISSIM microsimulation model to compare operational performances. The results showed that DDI outperformed traditional DIsig during peak hours. DDI performed better, providing better level of service, shorter delays, smaller queues, and higher throughput (Bared et al., 2005).

Similarly, numerous studies compared the operational performance of DIsig versus DDI. Hunter et al. (2019) used a calibrated VISSIM model to conduct a sensitivity analysis of operational performance under various lane configurations, traffic demands, and turn-movement ratios. Overall, the study found that a DIsig can be a preferred option at locations with traffic volumes well below capacity and cross-street left-turn proportions below 30% of the total cross-street demand. On the other hand, DDI was preferred at sites with traffic volumes near capacity and left-turn proportions from cross streets exceeding 50% of the total cross-street demand.

Almoshaogeh et al. (2020) compared the operational performance of DIsig and DDI using different crossover distances and various traffic volumes. It was found that DDI outperformed DIsig in terms of left-turn delay for all scenarios studied. In another study, Mogalli et al. (2019) found that DDI suffered a one-third control delay compared to DIsig.

Yeom et al. (2015) evaluated DDI performance following conversions from DIstop configurations using empirical data from Missouri and New York. Their findings showed substantial reductions in queue length: through-movement queues decreased from 19% to 83%, and left-turn queues decreased from 34% to 56%.

Several other studies (e.g., Tahmidul et al., 2022; Molan and Hummer, 2020; Molan et al., 2019) found that DDI outperformed DIsig, resulting in lower travel times and greater operational efficiency. Apart from this comparative research, some studies specifically addressed DDI-related elements, such as development of signal control schemes and optimization (e.g., Yang et al. 2014; Do et al., 2022; Warchol et al., 2017; Cunningham et al., 2016; Day et al., 2016; Day and Bullock, 2016; Warchol et al., 2017; Cheng et al., 2018), lane utilization (Yeom et al., 2014; Yeom et al., 2017), microsimulation modeling (Schroeder et al, 2014; Anderson et al., 2015; Leong et al., 2015), and control delay, capacity and queue spillback calculation (Xu et al., 2011; Pang et al., 2016; Warchol et al., 2016).

It is important to note that many studies (e.g., Molan, 2017; Tahmidul et al., 2022; Molan and Hummer, 2020; Molan et al., 2019) compared DDI and DIsig, but their primary focus was on evaluating newly proposed interchange designs against various alternatives. As a result, the comparison between DDI and DIsig was not the primary objective of these research studies. Nonetheless, the literature consistently indicates that DDI outperforms DIsig under high traffic demand and heavy turning traffic, while offering comparable performance in other traffic scenarios. However, challenges may arise with crossover or ramp terminals due to movements in DDI (Do et al., 2022), as through traffic in each direction must stop alternately, unlike traditional DI, which can accommodate concurrent through movements. As a result, DDI may not reduce delays during off-peak periods as effectively as traditional DI. Additionally, DIsig can operate with flashing yellow left-turns during low-volume periods of the day and night, further improving efficiency.

While numerous studies evaluated the operational performance between DIsig and DDI, limited attention has been paid to the transitions from DIstop to DIsig or DDI. One earlier study evaluating the conversion from all-way stop-controlled DI to DIsig found that standard signal warrants, such as those outlined in the MUTCD, may not adequately reflect the dynamics of ramp-terminal intersections (Chang and Messer, 1985; MUTCD, 2023). However, several studies dealt with individual performance of interchanges, such as different signal timing and phasing techniques, control strategy for closely spaced ramp terminals, and application of simulation methods for DIsig, mainly evaluating and improving its operational performance (e.g., Bertini et al., 2002; Tian et al., 2007; Koonce et al., 2007; Fang and Elefteriadou, 2006; Dorothy et al., 1998).

Interchange performance can vary significantly between peak, midday, and off-peak periods; such temporal variations must be considered when evaluating interchange alternatives. Overall, comprehensive comparative studies across DIstop, DIsig, and DDI are limited, especially those that incorporate a wide range of traffic conditions, time-of-day variations, and benefit-cost analyses.

Overall, the literature consistently showed that DDI outperformed DIstop or DIsig in many traffic scenarios, especially under high traffic volumes, imbalanced traffic volumes across different approaches, and with high left-turning traffic.

Several studies were conducted measuring the safety impacts of DDI. Six sites at Missouri were used to evaluate the safety implications of DDI that were converted from conventional DI using three safety analysis methods: before–after methods: naïve, empirical Bayes, and comparison group (Claros et al., 2015; Edara et al., 2015). Each method had different research approaches, however, all produced consistent results showing that converting a conventional DI to a DDI reduced crashes across all severities. The largest crash reductions, ranging from 59.3% to 63.2%, were observed for fatal and injury crashes. Property-damage-only crashes declined by 33.9% to 44.8%, and total crashes decreased by 40.8% to 47.9%. Further, it was found that DDI eliminated the left-turn angle crashes, which are common at conventional DI. Although wrong-way crashes are a potential concern for DDI, they represented only 4.8% of fatal and injury crashes at ramp terminals. Overall, these findings demonstrated that DDI provided substantial safety benefits.

Claros et al. (2015) developed crash CMFs for DDI ramp terminals using crash data from 20 sites in Missouri. The analysis revealed that fatal and injury crashes were reduced by 73.3% (using the comparison group method) and 63.4% (using the empirical Bayes method). DDI reduced the property-damage-only crashes by 21% (comparison group method) and 51.2% (empirical Bayes method), and total crashes by 42.7% (comparison group method) and 54.0% (empirical Bayes method). Based on empirical Bayes results, the authors recommended CMFs of 0.366 for fatal and injury crashes, 0.488 for property-damage-only crashes, and 0.460 for total crashes at DDI ramp terminals.

On the other hand, Hummer et al. (2016), using crash data from seven of the earliest DDI in the United States, also provided CMFs. Crash data were collected from four sites in Missouri, and the rest were from Kentucky, New York, and Tennessee. Based on the analysis, the authors recommended a CMF of 0.67 for all crashes. The CMF for injury crashes was 0.59, indicating larger safety benefits. Additional findings showed substantial reductions in angle and turning crashes, along with some reductions in rear-end crashes, although rear-end crashes remained the predominant crash type after DDI installations.

The potential impacts of DDI to its adjacent roadway facilities were also evaluated. Claros et al. (2017) used crash data from adjacent intersections and the speed-change lanes (SCLs) at the freeway entrances and exits of DDI facilities in Missouri. The empirical Bayes method did not find any statistically significant (positive or negative) impacts on the adjacent signalized intersections and SCLs due to the installation of DDI.

Nye et al. (2019) used national-level crash data to evaluate DDI's safety impacts. A comparison group method was used, based on 26 DDIs from 11 states, to derive a CMF of 0.633 for total crashes. Using the same method, the resulting CMFs were 0.441 for angle crashes, 0.549 for rear-end crashes, and 1.139 for sideswipe crashes. For fatal and injury crashes, the analysis produced a CMF of 0.461. Daytime and nighttime crashes yielded CMFs of 0.648 and 0.638**,** respectively. On the other hand, Abdelrahman et al. (2021) used even larger data sets to estimate the safety performance of DDI. This study collected samples of 80 DDIs from 24 states and 240 comparison sites of DI. As expected, the estimated CMFs indicated that replacing a conventional DI with a DDI resulted in notable crash reductions. The total, fatal-and-injury, rear-end, and angle or left-turn crashes were reduced by 14%, 44%, 11%, and 55%, respectively. Additionally, the safety performance functions developed in the study indicated that longer distances between crossovers or ramp terminals, as well as lower exit-ramp speed limits, were significantly associated with reduced crash frequency at DDI.

The literature reviewed above demonstrates that DDIs offer substantial safety benefits compared with DIstop or DIsig configurations. Although many other interchange forms may also enhance traffic performance, this review focused on DIstop, DIsig, and DDI designs, as these are the primary alternatives used in Nebraska.

2.4 Research Gap of Grade-Separated Interchanges

A literature review revealed that several studies have examined the safety implications of interchanges using appropriate statistical modeling approaches; however, predictive model-oriented studies comparing operational studies are relatively rare because most studies rely on graphical presentations to compare results across interchanges. Similar to intersections, predictive modeling for interchange-related studies is essential for informed interchange selection decisions, particularly across diverse traffic conditions. Therefore, this research aimed to develop data-driven predictive models for operational outcomes to support traffic agencies' interchange selection decisions. This project provided a framework and guidelines for interchange studies, based on comparative operational impacts and economic feasibility.

The literature review of interchanges revealed several gaps in research:

1. There is a lack of emphasis between DIstop and DIsig studies.
2. Most comparative studies focus solely on performance metrics without incorporating cost implications.
3. Simulation-based studies commonly calibrated models using mean values rather than full distributions of field performance, despite evidence that distribution-based calibration is crucial.
4. Overall, the findings may seem difficult to generalize in the literature due to variations in the interchanges studied, such as inconsistent lane and ramp configurations, and heterogeneity within diamond interchange design (either DIstop or DIsig), as well as within DDI design (e.g., yield or signalized ramp control).
5. There is no comprehensive, field-calibrated study that directly compares DIstop, DIsig, and DDI using a unified methodology, providing predictive models and guidelines for traffic agencies.

Chapter 3 Operational Studies of Unsignalized Intersections

One of the goals of this project was to evaluate the operational performance of unsignalized intersections along rural expressways in Nebraska.

The three primary objectives of this chapter are as follows.

1. Develop microsimulation models for unsignalized intersections and calibrate and validate the models using field-observed data.
2. Conduct comparative operational analysis among the intersections.
3. Develop statistical models for performance evaluation and comparison.

This chapter presents operational analyses that provide a critical foundation for the guidelines discussed in later sections of the report.

3.1 Modeling and Analysis of Traffic Operations

3.1.1 Site Characteristics

The research team collected operational data from nine sites. These included three TWSC intersections, three roundabouts, and three RCUTs, as presented in Figure 3.1.

Sites 1 to 3 are TWSC intersections. Sites 1 and 2 are located on two-lane highways, and Site 3 is located on four-lane highways.

Figure 3.1 Nine test sites in Nebraska for operational data collection (a) site 1: TWSC in the west of Avoca, (b) site 2: TWSC in the west of Mead , (c) site 3: TWSC in Hebron, (d) site 4: RCUT in Humphrey, (e) site 5: RCUT in North Bend, (f) site 6: RCUT in Murray, (g) site 7: Roundabout in Norfolk, (h) site 8: Roundabout in Kearney, and (i) site 9: Roundabout at Kearney

Site 1 (Figure 3.1a) is located in the west of Avoca, Nebraska. The major road approaches are Hwy 34 (E O St.) in the eastbound and westbound directions. The minor road is on Hwy 50 in the southbound and northbound directions and is stop-controlled at the intersection. All approaches have a 65-mph speed limit. This location has an AADT of 12,000 vehicles (NDOT AADT, 2025). At the intersection, the minor road has a wide single lane for through, left-turn, and right-turn movements in each approach. The westbound and eastbound traffic from the major approach has the following lane distribution properties: i) a single lane shares the through and right-turn movements, and ii) a dedicated left-turn bay for left-turn movement that yields to the opposing through traffic.

Site 2 (Figure 3.1b) is located in the west of Mead, Nebraska. The major road approaches are Hwy 77 and Hwy 92 (Co Rd M) in the eastbound and westbound directions. The minor road is on Hwy 77 (Co Rd 11) in the northbound and southbound directions and is stop-controlled at the intersection. One free right-turn ramp (FRT) starts at the major road from the east leg (600 feet from the intersection) and joins the minor road in the north leg (600 feet from the intersection). The other FRT ramp starts at the minor road from the north leg (650 feet from the intersection) and joins the major road in the west leg (700 feet from the intersection). All approaches have a 65-mph speed limit. This location has an AADT of around 21,500 vehicles per day.

At the intersection, the minor approach from the northbound traffic has a wide single lane for through, left-turn, and right-turn movements. On the other hand, the minor approach from the southbound traffic has the following properties: i) an FRT ramp for right-turn movements, and ii) a wide single lane for left-turn movements at the intersection.

The eastbound major approach has the following lane distribution: i) a single lane shares the through and right-turn movements, and ii) a dedicated left-turn bay for left-turn movements that yields to the opposing through traffic. The westbound traffic from the major approach has the following lane distribution properties: i) an FRT ramp for right turn movements, ii) a dedicated left-turn bay for left-turn movements that yield to the opposing through traffic, and iii) the through movement is free-flowing.

Site 3 (Figure 3.1c) is located at Hebron, Nebraska. The major approach on Hwy 81, a four-lane divided highway, forms the intersection with the minor approach on Hwy 136, a two-lane undivided highway. The major and minor approaches have speed limits of 70 mph and 65 mph, respectively. AADT is around 8,000 vehicles per day for this site. In each direction along the major road approach, there are two through lanes and one dedicated left- and right-turn lane. On the other hand, for the minor approach, each direction of traffic has one wide single lane for through, left- and right-turn movements, which are stop-controlled.

Sites 4, 5, and 6 (Figures 3.1d, 3.1e, 3.1f) are RCUTs located at Humphrey (Hwy 81/Hwy 91), North Bend (US 30/N 79), and Murray (Hwy 1, Murray Rd/US 75), Nebraska, respectively. The major approaches of these RCUTs are on four-lane divided highways, and the minor approaches are on two-lane divided highways. All the RCUTs have the same lane configuration. The major approach at the intersection has two through lanes and one dedicated lane for left- and right-turn movements. The left-turn at the major approach yields to the opposing major approach traffic. Both U-turn intersections are single-lane, yield-controlled, with a loon to facilitate truck turning movements. The minor approach has a single lane for right-turn movement only. At RCUT sites, the major road approach has a speed limit of 70 mph, and the minor approach speed limits range from 55 mph to 65 mph. The intersection AADTs for these sites ranged from 6,000 to 12,000 vehicles per day.

Sites 7, 8, and 9 (Figures 3.1g, 3.1h, 3.1i) are roundabouts located at Norfolk (US 275, N-35/Victory Rd) and Kearney (E 39th St/Kearney E Expy; E 39th St/Cherry Ave).

Both major and minor approaches of Site 7 at Norfolk are four-lane roads. The speed limits range from 45 mph to 55 mph. The roundabout AADT is around 13,500 vehicles per day. Within the roundabout, the advisory speed limit is 20 mph.

In the US 275 and N-35 section, the inner lane allows through and left-turn movements, and the outer lane allows through and right-turn movements. For the northbound traffic on Victory Rd. approach, the inner lane allows traffic movements in all directions, and the outer lane allows only right-turn movements.

On the other hand, for the southbound traffic on Victory Rd. approach, the inner lane allows through, and left-turn, and the outer lane allows through and right-turn movements.

Within the Norfolk roundabout, westbound and eastbound directions have two lanes. On the other hand, the northbound and southbound directions have single and two lanes, respectively.

The major approach of the roundabout (Site 8) at Kearney is on four-lane highways (northbound and southbound directions), and the minor approach (westbound and eastbound directions) is on two-lane highways. For both approaches of the major road, the inner lane allows through and left-turn movements and the outer lane allows through and right-turn movements. The major road's speed limit is 65 mph. The westbound and eastbound approaches of minor road have speed limits of 45 and 55 mph, respectively. On the other hand, for the minor approach, the single lane allows movement in all directions. The other roundabout at Site 9, a single-lane roundabout, is located adjacent to the roundabout at Site 8. The speed limit is 45 mph. The advisory speed limit for both roundabouts was 20 mph within the roundabout. The roundabouts at the Kearney sites experience an AADT of about 8,000 vehicles per day.

3.1.2 Operational Data Collection

Several sets of Miovision scouts (Miovision Scout, 2025) were used to collect traffic data at the 9 sites. In addition to Miovision, the research team used lidar speed guns to collect sample data on the approach speeds of vehicles entering, exiting, or turning within the intersection. Figure 3.2 shows the devices used for data collection.

a. Miovision b. Lidar speed gun

Figure 3.2 Data collection devices

Figure 3.1 also shows the locations of Miovision video camera units installed at each intersection to record traffic movements from August through October of 2022, 2023, and 2024, and January 2025. Video data collected using Miovision can be used to extract traffic demand, traffic composition, and travel time from one location of Miovision to others. However, Miovision is equipped with technology to collect media access control (MAC) addresses of devices installed or present in vehicles. Therefore, in addition to observing video data, MAC addresses were collected from Miovision as "objects" and these unique objects were matched using the R coding platform (Khattak, Haque, and Zhao, 2024; Haque and Sangster, 2018). This method was used to find travel times, volumes, and turn percentages. Note that this traffic information is available only from equipped vehicles. Furthermore, manual peak-hour traffic counts, including vehicle composition, were also performed. The corresponding information is summarized in Appendix A. On the other hand, the speed data were collected using Radar guns. This study used this data to develop a microsimulation model.

3.1.3 Microsimulation Model Development and Calibration Method

A microsimulation model needs proper calibration to reflect field observed driving behaviors and operational outcomes. There are several available traffic microsimulation tools for model development, including TransModeler™, AIMSUN™, TRANSIMS™, PARAMICS™, CORSIM™, and VISSIM™. A comparison of traffic modeling software tools is provided elsewhere (FHWA, 2016; Haque and Sangster, 2018).

All commercial microsimulation software packages have adjustable parameters to control internal driver behaviors. Even though they come with default parameters, better results are obtained when the parameters are calibrated to local field data (Haque et al., 2023a; Haque, 2022). The research team used VISSIM for this study due to its ability to model complex driver behaviors (PTV VISSIM, 2024; Haque et al, 2025; Haque et al., 2023b), its documented usage in Highway Capacity Manual (HCM) research, and suitability for near capacity analysis (HCM, 2022).

This research applied a robust calibration technique (Haque et al., 2023a) to develop the microsimulation model in three steps, as shown in Figure 3.3, for TWSC, RCUT, and roundabout intersections. The calibration goal was to ensure that the microsimulation accurately replicates the observed distribution of traffic performance, not just the mean, thereby producing a realistic model that imitates real-world traffic behaviors.

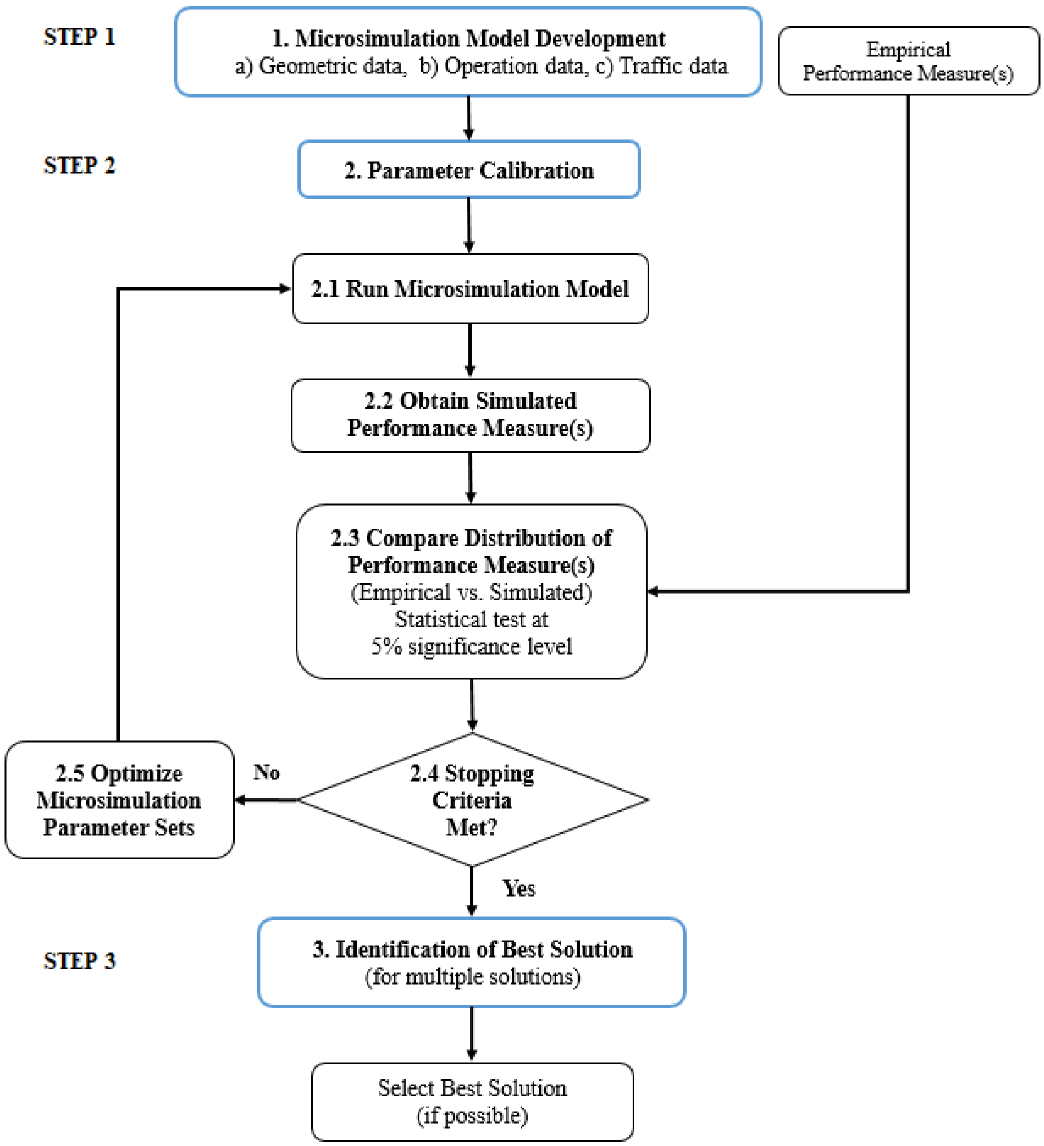


Figure 3.3 Microsimulation model calibration algorithm for intersection studies (Haque et al., 2023a)

A description of the three-step process is provided here.

Step 1. An important part of the model development is to obtain local data that represent the TWSC, RCUT, and roundabout intersections. This includes geometric data (e.g., horizontal curvature, segment lengths, grades), operational data (e.g., traffic control techniques, signage, speed limits), and traffic data (e.g., vehicle volumes, truck percentages).

For each intersection, two hours of empirical peak hour traffic data were used. The traffic demand, including the proportions of passenger cars (PC) and trucks (T) and their speed distributions, is a key simulation input. Simulated travel time from Step 1 was used to calibrate the microsimulation driving parameter in Step 2.

Step 2. Commercially available microsimulation models include parameters that users can adjust to control internal driver behavior (e.g., car-following, lane-changing, etc.) to represent field-observed traffic conditions. Default values of these parameters may be used; however, better results are obtained if the parameters are calibrated to local field data (Spiegelman et al., 2010).

This study used two types of parameters: car-following and gap-acceptance. For the car-following parameters, the Wiedemann 99 model was used for TWSC and RCUT. For roundabouts, the Wiedemann 74 model was deemed more appropriate, as vehicles on every approach yield to the circulating flow (Li et al., 2023; MassDOT, 2020). Three parameters from Wiedmann 74 were used: AX (average standstill distance), BX_add (additive part of the safety distance), and BX_mult (multiplicative part of the safety distance). For Wiedemann 99, seven parameters namely CC0 (standstill distance), CC1 (headway time), CC2 (following variation), CC3 (threshold for entering ‘‘following’’ mode), CC4 (negative following threshold), CC5 (positive following threshold), and CC6 (speed dependency of oscillation) were used. Detailed descriptions of these car-following parameters can be found elsewhere (PTV VISSIM, 2024).

For gap acceptance behavior, critical headway helps to accurately model traffic behavior at the intersection (HCM, 2022). VISSIM regulates these headway aspects using “priority rules” and “conflict area” functionalities (PTV VISSIM, 2024). Under “priority rules,” the “minimum gap time” parameter is tied to the critical headway.

Therefore, selected parameters from the car-following models and minimum gap time were calibrated for passenger cars and trucks so that the simulated performance measure (i.e., travel time) matched the field-observed data. This calibration process employed statistical tests (Kolmogorov–Smirnov [KS] test) to confirm that the simulated and field-observed performance were statistically similar (see Step 2.3 in Figure 3.3). Note that this methodology conforms to the FHWA microsimulation guidelines (FHWA, Traffic Analysis Toolbox Volume III, 2019).

Step 3. Several optimization algorithms may be used to find optimal solutions for the different parameter values tested in Step 2. These include the simplex method, the genetic algorithm, and simulated annealing (Kochenderfer et al., 2019). This study used genetic algorithms in MATLAB to find suitable parameter values that yield statistically similar simulated and field-observed results.

The calibration procedure may identify a zero solution, a single solution, or multiple solutions. If the optimization algorithm finds multiple parameters sets that satisfy the statistical test, an additional criterion is needed to determine the ‘‘best’’ parameter set. Therefore, the parameter set that resulted in the least error when the simulation results were compared to the observed data can be chosen, or the parameter set that best represents the local driver’s behavior can be selected, or standard guidelines such as the HCM can be used. In this project, video data were available to collect samples that represent headway parameters and minimum gap time. Therefore, the research team chose the final parameter solutions based on the calibration algorithm outcomes that aligned with field-observed behaviors.

3.1.4 Microsimulation Model Calibration Results

Using the algorithm in Figure 3.3, the calibration process used the travel time distributions of the eastbound left-turn movement at TWSC, the southbound through movement at RCUT, and the northbound left-turn movement at the roundabout for Sites 3, 5, and 8, respectively, as shown in Figure 3.1. The most critical movement, which is the left-turn for TWSC, was chosen because traffic from a minor road must stop and provide right-of-way to the major road traffic. On the other hand, for RCUT, the movement from minor-to-minor roads was chosen as it requires the use of the U-turn crossover (located 1200 feet from the main intersection). For the roundabout (with an inscribed circle diameter of around 210 feet), a regular left-turn from major to minor approach was chosen. Note that the length of these movements was around 1400, 4000, and 1100 feet, respectively, for TWSC, RCUT, and roundabout.

(a) (b) (c)

Figure 3.4 Test sites used for model calibration (a) site 3: eastbound left-turn movement at TWSC at Hebron, Nebraska, (b) site 5: southbound through movement at RCUT at North Bend, Nebraska, (c) site 8: northbound left-turn movement at roundabout at Kearney, Nebraska

Table 3.1 shows the final parameter values from peak hour period analysis.

Table 3.1 Calibrated simulation parameter values for unsignalized intersections

Calibrated Simulation Parameters Values								
		TWSC		**RCUT**		**Roundabout**		
	Parameters	**PC**	**T**	**PC**	**T**	**Parameters**	**PC**	**T**
Car-following behaviors	CC0 (ft)	10.0	12.0	10.0	12.0	AX	7.5	8.5
	CC1 (s)	*1.5 (0.2)	2.2 (0.1)	1.5 (0.2)	2.1 (0.4)	BX_add.	2.0	2.5
	CC2 (ft)	11.0	15.0	15.0	20.0	BX_mult.	4.0	4.5
	CC3-CC9	default	default	default	default			
Minimum gap acceptance (s)	Left-turn from major	5.1	7.0	5.1	7.0	All types of turns	3.6	4.8
	Right-turn from minor	6.1	7.5	6.1	7.5			
	Left-turn from minor	6.6	7.8	6.6	9.5			
	Through from minor	5.6	7.0	-	-			
	U-turn crossover	-	-	6.6	9.5			

*mean (standard deviation)

Table 3.2 Performance comparisons of unsignalized intersections using calibrated parameters

Performance Comparisons Using Calibrated Parameters							
		TWSC		**RCUT**		**Roundabout**	
	Parameters	**Emp.**	**Sim.**	**Emp.**	**Sim.**	**Emp.**	**Sim.**
Travel time statistics	Average travel time (s)	54.2	54.8	130.1	133.8	40.4	40.9
	Mean absolute error (%)	1.1		2.8		1.3	
	Welch T-test (p-value)	0.85		0.73		0.79	
	KS-test (p-value)	0.08		0.31		0.07	

Table 3.2 presents the field-observed average travel times for TWSC, RCUT, and roundabout for the selected movements, which were 54.2, 130.1, and 40.4 seconds, respectively. The calibrated simulation models reported travel times of 54.8, 133.8, and 40.9 seconds, corresponding to differences of 1.1%, 2.8%, and 1.3% from sites, indicating simulated travel times closely aligned with field observations. Both the Welch t-test and the KS test yielded p-values exceeding 0.05, indicating that there was no statistically significant difference between the simulated and empirical values at 95% confidence level. Furthermore, the KS test indicated that the distributions of simulated and observed travel times were statistically similar. Therefore, the simulation model replicated real-world traffic performance variations and was ready for a sensitivity analysis.

3.1.5 Sensitivity Analysis Framework

This study conducted a comprehensive sensitivity analysis for three intersection types using the calibrated microsimulation model. For each intersection, the following combination of 1,792 traffic scenarios was simulated by varying:

- Major-road AADT over seven levels (1,500; 2,500; 5,000; 10,000; 15,000; 20,000; 25,000),
- Minor-road AADT over four levels (500; 2,500; 5,000; 7,500),
- Turning movement distributions (left/through/right) in percentages of 5/90/5, 20/70/10, 40/40/20, and 60/10/30 for both major and minor approaches, and
- Time-of-day (TOD) for four periods:
    - Peak: two 2-hour windows (e.g., 7–9 am and 4–6 pm),
    - Midday peak (e.g., 11am-1pm)
    - Off-peak daytime (Off-Peak 1): all remaining hours between 6 am-midnight, and

- Off-peak nighttime (Off-Peak 2): midnight–6 am.

Based on observed traffic volume patterns at nine sites and the NDOT "Annual Traffic Count Data" report (NDOT ATC, 2025), each TOD period was assigned a representative AADT proportion. These detailed traffic periods were essential both for performance comparisons and subsequent benefit-cost analysis.

For each of the three intersection types, 1,792 traffic scenarios (a total of 5,376) were simulated 10 times with different random seeds and appropriate warm-up periods, resulting in 53,760 total simulation runs. Each run produced stopped delay and vehicular delay for left-turn, through, and right-turn movements from four directions. Then, delays were aggregated for the major and minor approaches and for the overall intersection. This process yielded 38 delay datapoints per run, totaling over two million. The entire workflow of simulation execution, data collection, and processing utilized a combination of full and partial automation.

For a fair comparison, all intersections' lane distributions were kept the same, except for the fundamental geometric differences among them. In this case, four-lane expressways (major approach with 70 mph speed limit) intersect two-lane highways (minor approach with 65 mph speed limit). For each direction, the major and minor approaches have two and one through lanes, respectively, and TWSC and RCUT have one dedicated left-turn and one right-turn lane. Within the roundabout (20 mph speed limit), there are two lanes circulating to connect both sides of the major road, and one lane circulating to connect both sides of the minor road. The minor approaches of both TWSC and RCUT, and the two U-turn crossovers of RCUT, are stop-controlled, and the major left-turn at the intersection yields to the opposing through traffic.

The traffic stream includes 10% trucks at all intersections. This truck proportion is based on observed truck traffic. Midwestern states typically experience higher proportions of truck traffic compared with many other regions (Zhao et al., 2025).

This sensitivity framework supports the intersection comparison and selection models presented in the rest of the chapter.

3.2 Performance Evaluation and Predictive Modeling

3.2.1 Comparative Performance Evaluation

A subset of the sensitivity results is shown in Figures 3.5, 3.6, and 3.7, representing the peak hour, all AADT combinations, and varying left-turn percentages (four and two levels for major and minor approaches) for TWSC, roundabout, and RCUT, respectively. The average intersection delays are listed as HCM defined LOS (6). The LOS thresholds are included in Appendix B.

			Minor Road AADT							
			500		2500		5000		7500	
B/C Analysis		Left Turn %	5	60	5	60	5	60	5	60
Major Road AADT	1500	5	A	A	A	A	B	A	B	B
		20	A	A	A	A	B	A	B	B
		40	A	A	A	A	B	A	B	B
		60	A	A	A	A	B	A	C	B
	2500	5	A	A	A	A	B	A	C	B
		20	A	A	A	A	B	A	C	B
		40	A	A	A	A	B	A	C	B
		60	A	A	A	A	B	A	C	B
	5000	5	A	A	A	A	B	A	C	B
		20	A	A	A	A	B	A	D	B
		40	A	A	A	A	B	A	D	B
		60	A	A	A	A	B	A	E	B
	10000	5	A	A	A	A	D	B	F	F
		20	A	A	A	A	E	C	F	F
		40	A	A	A	A	F	C	F	F
		60	A	A	B	A	F	D	F	F
	15000	5	A	A	B	A	F	F	F	F
		20	A	A	C	B	F	F	F	F
		40	A	A	F	D	F	F	F	F
		60	B	A	F	F	F	F	F	F
	20000	5	A	A	E	E	F	F	F	F
		20	A	A	F	F	F	F	F	F
		40	C	C	F	F	F	F	F	F
		60	F	E	F	F	F	F	F	F
	25000	5	A	A	F	F	F	F	F	F
		20	C	C	F	F	F	F	F	F
		40	F	F	F	F	F	F	F	F
		60	F	F	F	F	F	F	F	F

Figure 3.5 LOS of selected scenarios for TWSC for peak hour periods

			Minor Road AADT							
			500		2500		5000		7500	
B/C Analysis		Left Turn %	5	60	5	60	5	60	5	60
Major Road AADT	1500	5	A	A	A	A	A	A	A	A
		20	A	A	A	A	A	A	A	A
		40	A	A	A	A	A	A	A	A
		60	A	A	A	A	A	A	A	A
	2500	5	A	A	A	A	A	A	A	A
		20	A	A	A	A	A	A	A	A
		40	A	A	A	A	A	A	A	A
		60	A	A	A	A	A	A	A	A
	5000	5	A	A	A	A	A	A	A	A
		20	A	A	A	A	A	A	A	A
		40	A	A	A	A	A	A	A	A
		60	A	A	A	A	A	A	A	A
	10000	5	A	A	A	A	A	A	A	A
		20	A	A	A	A	A	A	A	A
		40	A	A	A	A	A	A	B	B
		60	A	A	A	A	B	A	C	B
	15000	5	A	A	A	A	A	A	B	B
		20	A	A	A	A	A	A	B	B
		40	A	A	A	A	B	B	C	C
		60	B	B	C	C	E	C	F	E
	20000	5	A	A	A	A	A	A	B	C
		20	A	A	A	A	B	B	C	C
		40	B	B	C	C	E	C	F	E
		60	F	F	F	F	F	F	F	F
	25000	5	A	A	A	A	B	B	C	D
		20	A	A	B	A	C	B	E	D
		40	D	D	F	E	F	F	F	F
		60	F	F	F	F	F	F	F	F

Figure 3.6 LOS of selected scenarios for roundabout for peak hour periods

			Minor Road AADT							
			500		2500		5000		7500	
B/C Analysis		Left Turn %	5	60	5	60	5	60	5	60
Major Road AADT	1500	5	A	A	C	B	C	C	D	D
		20	A	A	C	B	C	C	D	D
		40	A	A	C	B	C	C	D	C
		60	A	A	C	B	C	C	D	C
	2500	5	A	A	B	B	C	C	D	C
		20	A	A	B	B	C	C	D	C
		40	A	A	B	B	C	C	D	C
		60	A	A	B	B	C	C	D	C
	5000	5	A	A	B	B	C	B	C	C
		20	A	A	B	B	C	B	C	C
		40	A	A	B	B	C	B	C	C
		60	A	A	B	B	C	B	C	C
	10000	5	A	A	A	A	B	B	C	C
		20	A	A	B	A	B	B	C	C
		40	A	A	B	A	B	B	C	C
		60	A	A	B	A	C	B	C	C
	15000	5	A	A	A	A	B	B	D	E
		20	A	A	A	A	B	B	C	D
		40	A	A	B	A	C	B	C	C
		60	A	A	B	B	C	B	D	C
	20000	5	A	A	A	A	C	C	E	F
		20	A	A	B	A	C	B	D	E
		40	A	A	B	B	C	C	E	C
		60	B	B	C	B	E	C	F	D
	25000	5	A	A	B	A	D	D	F	F
		20	A	A	B	B	C	C	F	F
		40	B	B	C	C	F	E	F	F
		60	D	D	F	F	F	F	F	F

Figure 3.7 LOS of selected scenarios for RCUT for peak hour periods

Figure 3.5 shows that for up to 5,000 AADT for both approaches, TWSC's LOS remained within the A-B range. After 10,000 major-AADT and higher level of minor-AADT, LOS changed to D-F. As major AADT increased, minor road delays increased, as fewer gaps were available for merging or crossing.

The roundabout in Figure 3.6 was mostly LOS A up to 10,000 major-AADT. For 15,000 AADT and beyond, and with higher minor left-turn volumes, the LOS shifted to range B-F. In general, RCUT maintained LOS A-C, up to 20,000 major-AADT and 2,500 minor-AADT as shown in Figure 3.7. However, beyond this threshold, LOS changed to the C-F range. Note that RCUT delays considered the extra distance travel time for minor left-through traffic using U-turn crossovers as recommended by HCM (HCM, 2022). Compared to the 50-second delay threshold for LOS F in regular unsignalized intersections, RCUT used an 80-second threshold per HCM (HCM, 2022) to account for the extra distance.

Notably, high left-turning traffic volume from major roads contributed to reduced LOS for all intersections. For minor approaches, increasing left-turn share from 5% to 60% results in a decrease in the left-through percentage from 95% to 70%, thereby improving LOS in some scenarios for TWSC and RCUT by reducing conflicting interactions.

Note that the LOS concept is based on drivers' experiences with intersection operations. While non-peak-hour traffic demand is lower, traffic agencies often tolerate LOS F during peak hours if queues do not block nearby intersections or if queue spillback does not occur frequently. Therefore, intersection conversion decisions are not based solely on LOS but also on a comprehensive assessment of delay magnitude, TOD conditions, safety, and costs. Nonetheless, LOS is a key aspect in intersection preference decisions.

3.2.2 Predictive Performance Model

A series of log-linear regression models was developed to quantify and understand the factors influencing intersection delays across thousands of scenarios. Equation (3.1) shows the general form of the model.

$$\log(D) = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \ldots\ldots + \beta_k X_k \tag{3.1}$$

Here, D is the estimated average intersection delay in sec/veh., β_0 is the intercept, and β_1, $\beta_2, \ldots, \beta_k$ are the estimated coefficients of the predictors $X_1, X_2, \ldots, X_k$. The predictors are major-road AADT (AADT_M) and minor-road AADT (AADT_C) in thousands, left-turn percentage from major road (LT_M) and minor road (LT_C), and TODs for peak hour (TOD_PEAK), midday peak (TOD_MID), off-peak hour 1 (TOD_OFF1), and off-peak hour 2 (TOD_OFF2).

Note that through and right-turns are excluded from the model to prevent multicollinearity, as their percentages are directly associated with the left-turn percentage, which sums to 100%.

The log-linear model was applied because the corresponding delays were found to be right-skewed, with higher traffic demand or left-turn (and through movement) causing substantial delay compared to its lower-delay counterpart. Also, log-linear models will not predict negative delays, unlike linear regression, and provide a better fit.

Table 3.3shows model outcomes for peak, midday peak, and all TODs, including estimated coefficients, standard errors, significance, and corresponding model fits for all intersections.

Table 3.3 Log-linear regression models for average intersection delay for peak, midday peak, and all TOD periods

Intersections	Parameters	Peak Hours			Midday Peak Hours			All TODs		
		Est.	SE	p val.	Est.	SE	p val.	Est.	SE	p val.
TWSC	Intercept	-0.236	0.114	0.039	-0.523	0.081	0.000	0.760	0.065	0.000
	AADT_M	0.121	0.114	0.000	0.100	0.003	0.000	0.056	0.002	0.000
	AADT_C	0.384	0.014	0.000	0.383	0.009	0.000	0.337	0.007	0.000
	LT_M	0.013	0.001	0.000	0.011	0.001	0.000	0.010	0.001	0.000
	LT_C	-0.003	0.001	0.063	-0.003	0.001	0.004	-0.002	0.001	0.002
	TOD_PEAK	-	-	-	-	-	-	0*	-	-
	TOD_MID	-	-	-	-	-	-	-0.602	0.052	0.000
	TOD_OFF1	-	-	-	-	-	-	-1.330	0.052	0.000
	TOD_OFF2	-	-	-	-	-	-	-2.548	0.052	0.000
	Adjusted R²	*0.777*			*0.852*			*0.762*		
Roundabout	Intercept	-0.574	0.079	0.000	-0.261	0.054	0.000	0.579	0.040	0.000
	AADT_M	0.103	0.003	0.000	0.064	0.002	0.000	0.054	0.001	0.000
	AADT_C	0.206	0.011	0.000	0.177	0.007	0.000	0.151	0..4	0.000
	LT_M	0.025	0.001	0.000	0.015	0.001	0.000	0.012	0.001	0.000
	LT_C	-	-	-	-	-	-	-	-	-
	TOD_PEAK	-	-	-	-	-	-	0*	-	-
	TOD_MID	-	-	-	-	-	-	-0.533	0.030	0.000
	TOD_OFF1	-	-	-	-	-	-	-0.956	0.036	0.000
	TOD_OFF2	-	-	-	-	-		-1.966	0.036	0.000
	Adjusted R²	*0.770*			*0.769*			*0.765*		
RCUT	Intercept	1.227	0.09	0.000	1.682	0.049	0.000	2.059	0.043	0.000
	AADT_M	0.028	0.003	0.000	0.018	0.001	0.000	0.024	0.001	0.000
	AADT_C	0.296	0.009	0.000	0.246	0.006	0.000	0.266	0.004	0.000
	LT_M	0.008	0.001	0.000	0.004	0.001	0.000	0.004	0.0005	0.000
	LT_C	-0.003	0.001	0.000	-0.003	0.001	0.000	-0.003	0.0005	0.000
	TOD_PEAK	-	-	-	-	-	-	0*	-	-
	TOD_MID	-	-	-	-	-	-	-0.394	0.034	0.000
	TOD_OFF1	-	-	-	-	-	-	-0.596	0.034	0.000
	TOD_OFF2	-	-	-	-	-	-	-1.021	0.034	0.000
	Adjusted R²	*0.702*			*0.800*			*0.719*		

Table 3.3 offers quick references for practitioners to delay predictions for thousands of scenarios. For example, for all TODs, roundabout's coefficient of AADT_M, AADT_C, and LT_M are positive, meaning that increasing these factors contributes to higher delays. To be precise, every unit (1,000 AADT) increase of major-AADT, provided other variables are constant, increased the intersection delay by 5.5% (coefficient =0.054). Similarly, a unit increase in minor-AADT and a 1% increase in major left-turn traffic resulted in 16.3% and 1.2% higher delays, respectively. On the other hand, TOD effects relative to peak hour showed considerably lower delays during midday, daytime off-peak, and nighttime off-peak, by around 41%, 62%, and 86%, respectively. Note that all these changes were statistically significant, indicated by the corresponding p-values.

Table 3.4 Log-linear regression models for average intersection delay for different major AADT ranges for peak hour periods

Intersections	Parameters	Major AADT (1500 to 5000)			Major AADT (5000 to 15000)			Major AADT (15000 to 25000)		
		Est.	SE	p val.	Est.	SE	p val.	Est.	SE	p val.
TWSC	Intercept	0.520	0.067	0.000	-1.367	0.142	0.000	0.789	0.379	0.038
	AADT_M	0.027	00.013	0.042	0.198	0.009	0.000	0.057	0.017	0.001
	AADT_C	0.313	0.007	0.000	0.530	0.014	0.000	0.365	0.026	0.000
	LT_M	0.003	0.0009	0.001	0.013	0.001	0.000	0.021	0.003	0.000
	LT_C	-0.003	0.0009	0.000	-0.006	0.001	0.001	-	-	-
	Adjusted R²	*0.906*			*0.905*			*0.856*		
Roundabout	Intercept	0.283	0.036	0.000	-0.145	0.072	0.045	-2.11	0.196	0.000
	AADT_M	0.034	0.007	0.000	0.084	0.004	0.000	0.148	0.008	0.000
	AADT_C	0.244	0.004	0.000	0.186	0.007	0.000	0.179	0.013	0.000
	LT_M	0.002	0.0005	0.000	0.015	0.0009	0.000	0.052	0.001	0.000
	LT_C	-	-	-	-0.001	0.0009	0.136	-0.003	0.001	0.072
	Adjusted R²	*0.940*			*0.864*			*0.882*		
RCUT	Intercept	2.272	0.064	0.000	1.361	0.054	0.000	-1.198	0.182	0.000
	AADT_M	0.122	0.014	0.000	-	-	-	0.124	0.007	0.000
	AADT_C	0.253	0.007	0.000	0.290	0.007	0.000	0.339	0.012	0.000
	LT_M	-	-	-	0.003	0.0009	0.001	0.019	0.001	0.000
	LT_C	-0.003	0.0009	0.002	-0.002	0.0009	0.006	-0.003	0.001	0.029
	Adjusted R²	*0.854*			*0.890*			*0.858*		

Table 3.4 provides log-linear regression model results for peak-hour periods for major-AADT ranges of 1,500 to 5,000, 5,000 to 15,000, and 15,000 to 25,000 for all intersections. The purpose of Table 3.4 is to present a more accurate predictive model (with higher R^2 values than in Table 3.3), given the importance of peak periods discussed earlier.

3.2.3 Performance-Based Intersection Choice Model

This study developed a series of binary logistic regression models to examine the likelihood of selecting a given intersection from pair-wise comparison. Logistic regression models the log-odds of a binary outcome (i.e., intersection with lesser delay in this case) as a linear function of predictor variables. Equation (3.2) shows the general form of the model.

$$\log\left(\frac{p}{1-p}\right) = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \ldots\ldots + \beta_k X_k \tag{3.2}$$

Here, p is the probability of selecting a given type of intersection. The associated predictors are the same as in Equation (3.1).

Table 3.5 Logistic regression models for intersection choice for peak and all TOD periods based on intersection delay

Intersection Groups	Parameters	Peak Hours			All TODs		
		Est.	SE	p val.	Est.	SE	p val.
Model 1 TWSC vs Roundabout	Intercept	1.767	0.378	0.000	1.005	0.252	0.000
	AADT_M	-0.108	0.017	0.000	-0.163	0.012	0.000
	AADT_C	0.582	0.075	0.000	1.267	0.074	0.000
	LT_M	-0.020	0.006	0.000	-0.012	0.004	0.001
	LT_C	-	-	-	-	-	-
	TOD_PEAK	-	-	-	0*	-	-
	TOD_MID	-	-	-	-0.029	0.240	0.904
	TOD_OFF1	-	-	-	-0.205	0.242	0.396
	TOD_OFF2	-	-	-	-1.774	0.247	0.000
	AIC	325.178			903.954		
	McFadden R²	*0.298*			*0.545*		
	Cox–Snell R²	*0.260*			*0.448*		
	Nagelkerke R²	*0.409*			*0.675*		
	Cor. Pred. %	*84.150*			*92.080*		
Model 2 TWSC vs RCUT	Intercept	-4.277	0.511	0.000	-7.659	0.542	0.000
	AADT_M	0.255	0.021	0.000	0.391	0.023	0.000
	AADT_C	0.171	0.053	0.001	0.396	0.045	0.000
	LT_M	0.015	0.006	0.022	0.046	0.005	0.000
	LT_C	-0.003	-0.006	0.620	-0.007	0.004	0.149
	TOD_PEAK	-	-	-	0*	-	-
	TOD_MID	-	-	-	-1.167	0.251	0.000
	TOD_OFF1	-	-	-	-4.180	0.345	0.000
	TOD_OFF2	-	-	-	-23.962	598.35	0.968
	AIC	*343.487*			*632.251*		
	McFadden R²	*0.440*			*0.683*		
	Cox–Snell R²	*0.453*			*0.523*		
	Nagelkerke R²	*0.607*			*0.790*		
	Cor. Pred. %	*87.720*			*94.920*		
Model 3 RCUT vs Roundabout	Intercept	9.876	1.122	0.000	5.223	0.412	0.000
	AADT_M	-0.440	0.051	0.000	-0.331	0.020	0.000
	AADT_C	0.703	0.109	0.000	0.827	0.059	0.000
	LT_M	-0.121	0.163	0.000	-0.058	.005	0.000
	LT_C	-0.006	0.009	0.472	-	-	-
	TOD_PEAK	-	-	-	0*	-	-
	TOD_MID	-	-	-	0.840	0.263	0.001
	TOD_OFF1	-	-	-	1.828	0.290	0.000
	TOD_OFF2	-	-	-	2.539	0.316	0.000
	AIC	*174.260*			*682.962*		
	McFadden R²	*0.673*			*0.594*		
	Cox–Snell R²	*0.531*			*0.420*		
	Nagelkerke R²	*0.786*			*0.700*		
	Cor. Pred. %	*90.400*			*91.630*		

Note: *This coefficient is redundant in the model and set as zero.

Est. (Estimated), SE (Standard Error), p val. (p value), Cor. Pred. % (Correct Prediction %)

Table 3.5 shows the results of the logistic regression for Model 1 (probability of selecting roundabout over TWSC), Model 2 (probability of selecting RCUT over TWSC), and Model 3 (probability of selecting roundabout over RCUT) based on intersection delay for peak and all TOD periods.

Using the results from Table 3.5, Equation (3.3) shows an example application of the Model 2 logistic regression for different TODs.

$$\log\left(\frac{P_{\text{Model 2}}}{1-P_{\text{Model 2}}}\right) = -7.659 + (0.391\times AADT\_M) + (0.396\times AADT\_C) + (0.046\times LT\_M) - (0.007\times LT\_C) + (0\times TOD\_PEAK) - (1.167*TOD\_MID) - (4.180*TOD\_OFF1) - (23.962*TOD\_OFF2) \tag{3.3}$$

Where,

$P_{\text{Model 2}}$ = probability of choosing an RCUT over TWSC

AADT_M = major-road AADT (in thousands); application range 1,500-25,000

AADT_C = minor-road AADT (in thousands); application range 500-7,500

LT_M = major road left-turning traffic percentage; application range 5%-60%

LT_C = minor road left-turning traffic percentage; application range 5%-60%

TOD_PEAK = 0 (other TOD coefficients are estimated relative to the peak period)

TOD_MID = 1 for midday period; 0 otherwise

TOD_OFF1 = 1 for off peak 1 period; 0 otherwise

TOD_OFF2 = 1 for off peak 2 period; 0 otherwise

The coefficients for AADT_M and AADT_C in Equation (3.3) are positive and equal to 0.391 and 0.396, respectively, indicating that each unit increase in AADT increases the odds of selecting RCUT over TWSC by 48% and 49%, holding all other variables constant. Similarly, the midday peak and daytime off-peak reduce the odds of choosing the RCUT by 69% and

98.5%, respectively, indicating its reduced benefit in these TODs compared to the peak period, likely because the added travel time is less justified under low traffic volumes.

A predicted probability of 0.5 or higher indicates preference for the corresponding alternate intersections. Using this threshold, Table 3.5 lists the correct prediction rates for Models 1, 2, and 3 as 92.08%, 94.92%, and 91.63%, respectively, across all TODs.

Chapter 4 Operational Studies of Interchanges

One of the primary goals of this project was to evaluate the operational performance of interchanges in Nebraska. Three interchange types, DIstop, DIsig, and DDI, were analyzed and compared. Thousands of traffic scenarios were simulated using calibrated microsimulation models, and statistical models were developed to estimate performance outcomes and determine the preferred interchange type under different traffic conditions.

The three major objectives of this chapter are as follows.

1. Develop microsimulation models for interchanges and calibrate and validate the models using field-observed data.
2. Conduct a comparative operational analysis among the intersections.
3. Develop statistical models for performance evaluation and comparison.

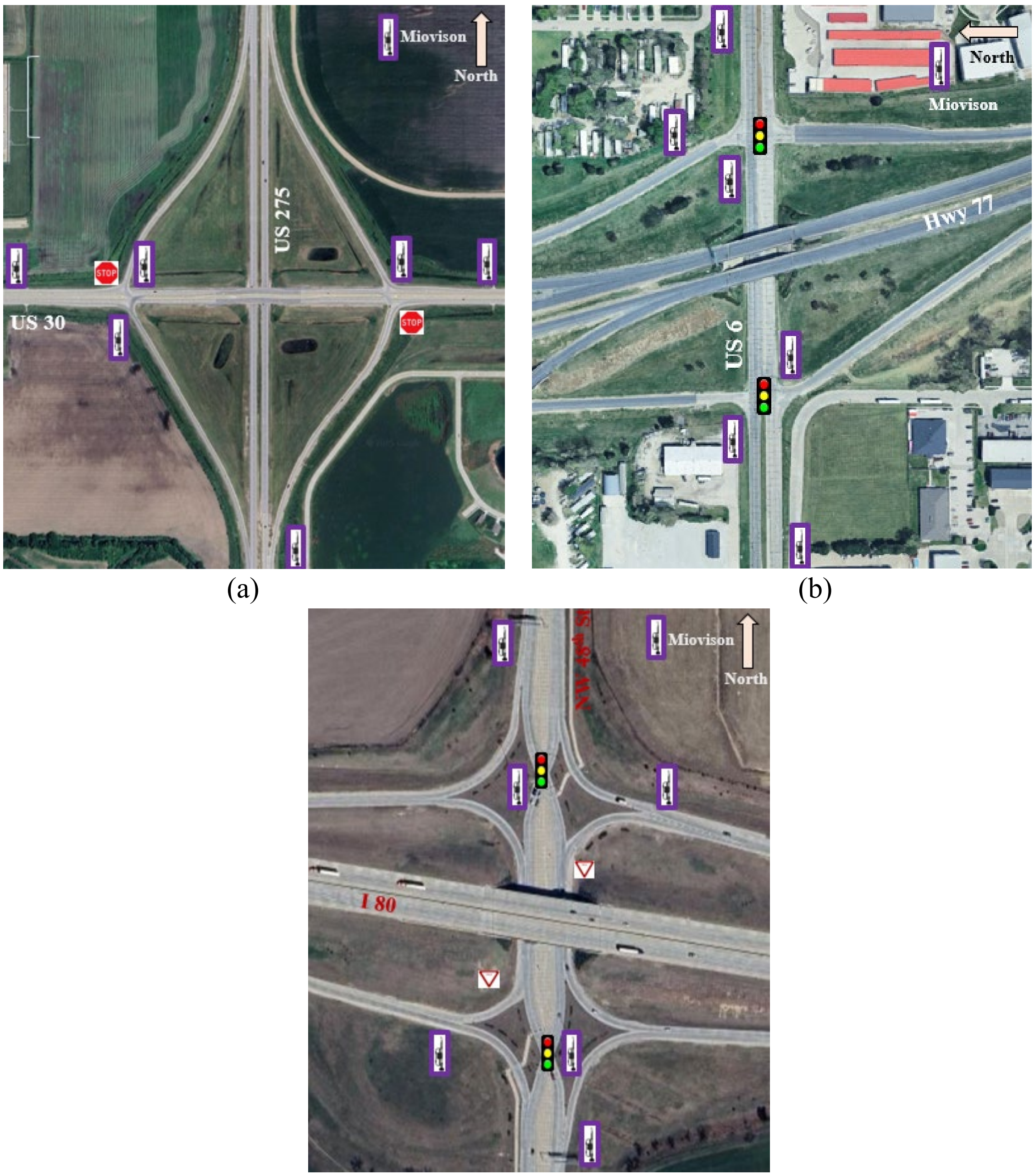
This chapter presents the operational studies that formed a key component of the interchange guidelines developed later in the report.

4.1 Modeling and Analysis of Traffic Operations

4.1.1 Site Characteristics

The research team collected operational data from three sites for a top-controlled diamond interchange (DIstop), signal-controlled diamond interchange (DIsig), and DDI. Figure 4.1 shows the three interchanges.

DIstop (site 10, shown in Figure 4.1a) is located in Fremont, Nebraska, connecting US 275 (a four-lane expressway) and US 30 (a four-lane highway). Ramps from US 275 connect to the cross-street at US 30 through stop controls. The northbound ramp has a single lane with a right-turn bay, whereas the southbound ramp has all movements in a wide single lane near the terminals. Each terminal of the cross-street approach has two through lanes, one left-turn bay, and one right-turn lane. The ramps and cross-street speed limits are 55 mph and 45 mph, respectively.



(a) (b)

(c)

Figure 4.1 Three Nebraska test sites for traffic operations data collection (a) Site 10: stop-controlled diamond interchange at Fremont, (b) Site 11: signalized diamond interchange at Lincoln, (c) Site 12: diverging diamond interchange at Lincoln

DIsig (site 11, shown in Figure 4.1b) is located in Lincoln, Nebraska, connecting Hwy 77 (four-lane expressway) and US 6 (four-lane highway). Ramps from Hwy 77 connect to the cross-street at US 6 through signal controls. The southbound ramp has a single-wide lane that facilitates movement in all directions in the terminal. On the other hand, the northbound ramp has a single lane, but near the terminal it has three bay lanes: one for right-turn-only movement, one for left-turn-only movement, and one for shared through and left-turn movements. Each terminal of the cross-street approach has two through lanes, a left-turn bay, and a right-turn lane. The ramps and cross-street speed limits are 50 mph and 45 mph, respectively.

DDI (site 12, shown in Figure 4.1c) was located in Lincoln, Nebraska, connecting I-80 and NW 48th St. Single-lane ramps from I-80 connect to the cross-street at NW 48th. Left-turn ramps are yield-controlled near the terminals. At signal-controlled terminals, the cross street has two lanes in each direction. Within the terminals, for both directions, there is a left-turn lane for traffic to navigate to the on-ramp. The ramps and cross-street speed limits are 45 mph and 30 mph, respectively.

4.1.2 Operational Data Collection

Traffic operations data were collected from three sites across Nebraska as shown in Figure 4.1.

The speed limit on all freeways/expressways was within 60 to 70 miles per hour (mph), and the single-lane off-ramp speed limit ranged from 45 to 55 mph. Near the ramp intersections, the cross-street speed limits were 45 mph for both DIstop and DIsig, and 30 mph for DDI. The cross-streets were located on four-lane roads. The AADT ranged from 11,000 to 22,000 for the cross-street and from 900 to 6,000 for the ramp.

Note that the left-turning off-ramp traffic approaching the cross-street at DDI is yield-controlled, as shown in Figure 4.1, along with other stop and signal controls depicted for interchanges.

Figure 4.1 also shows the locations of video camera units (Miovision Scout, 2025) installed at each intersection to record traffic movements from September 2023 and April-May 2024. In addition to recording video footage, the unit collected media access control (MAC) addresses of devices present in the vehicles, which were used to extract traffic demand, traffic composition, and travel time from one location to another (i.e., from one video camera to another) using R language-based algorithms as discussed in Chapter 3. Furthermore, manual peak-hour traffic counts, including vehicle composition, were also performed. The corresponding information was summarized in Appendix A.

The speed data were collected using Lidar speed guns. This research used the collected data to develop the microsimulation model discussed in the next section.

4.1.3 Microsimulation Model Development and Calibration Method

This research utilized a robust calibration technique (Haque et al., 2023a) to develop the microsimulation model in three steps as shown in Figure 3.3 in Chapter 3 for DIstop, DIsig, and DDI. The three steps of the calibration algorithm are similar but adjusted for interchange studies. As usual, the goal of the calibration was to ensure that the microsimulation accurately replicates the observed distribution of traffic performance, not just the mean, thereby providing a realistic model that imitates real-world traffic behaviors.

Step 1 used field data (e.g., geometric, operational, traffic, and signal) for microsimulation model development. For each interchange, two hours of empirical peak-hour traffic data were used. The traffic demand, including the passenger car (PC) and truck (T) proportions and speed distributions, is a key simulation input. Simulated travel time from Step 1 was used to calibrate the microsimulation driving parameter in Step 2.

All commercial microsimulation software packages include adjustable parameters to control internal driver behavior. Even though they have default parameters, better results are obtained when the parameters are calibrated to local field data (Haque et al., 2023a; Haque, 2022; Zhao et al., 2022). This research used VISSIM due to its ability to model complex driver behaviors and traffic scenarios for regular and unconventional interchanges (PTV VISSIM, 2024; Haque et al., 2024; Haque et al., 2023b; Yang et al., 2018).

This study used two types of parameters for interchange modeling: car-following and gap acceptance. For the car-following parameters, the Wiedemann 99 model was used for DIstop. For DIsig and DDI, the Wiedemann 74 model was deemed more appropriate as vehicles are required to stop at signalized ramp terminals (Li et al., 2013; MassDOT, 2019). Three parameters from Wiedmann 74 were used: AX (average standstill distance), BX_add (additive part of the safety distance), and BX_mult (multiplicative part of the safety distance). For Wiedemann 99, seven parameters, namely CC0 (standstill distance), CC1 (headway time), CC2 (following variation), CC3 (threshold for entering ''following'' mode), CC4 (negative following threshold), CC5 (positive following threshold), and CC6 (speed dependency of oscillation), were used. Detailed descriptions of these car-following parameters can be found elsewhere (PTV VISSIM, 2024).

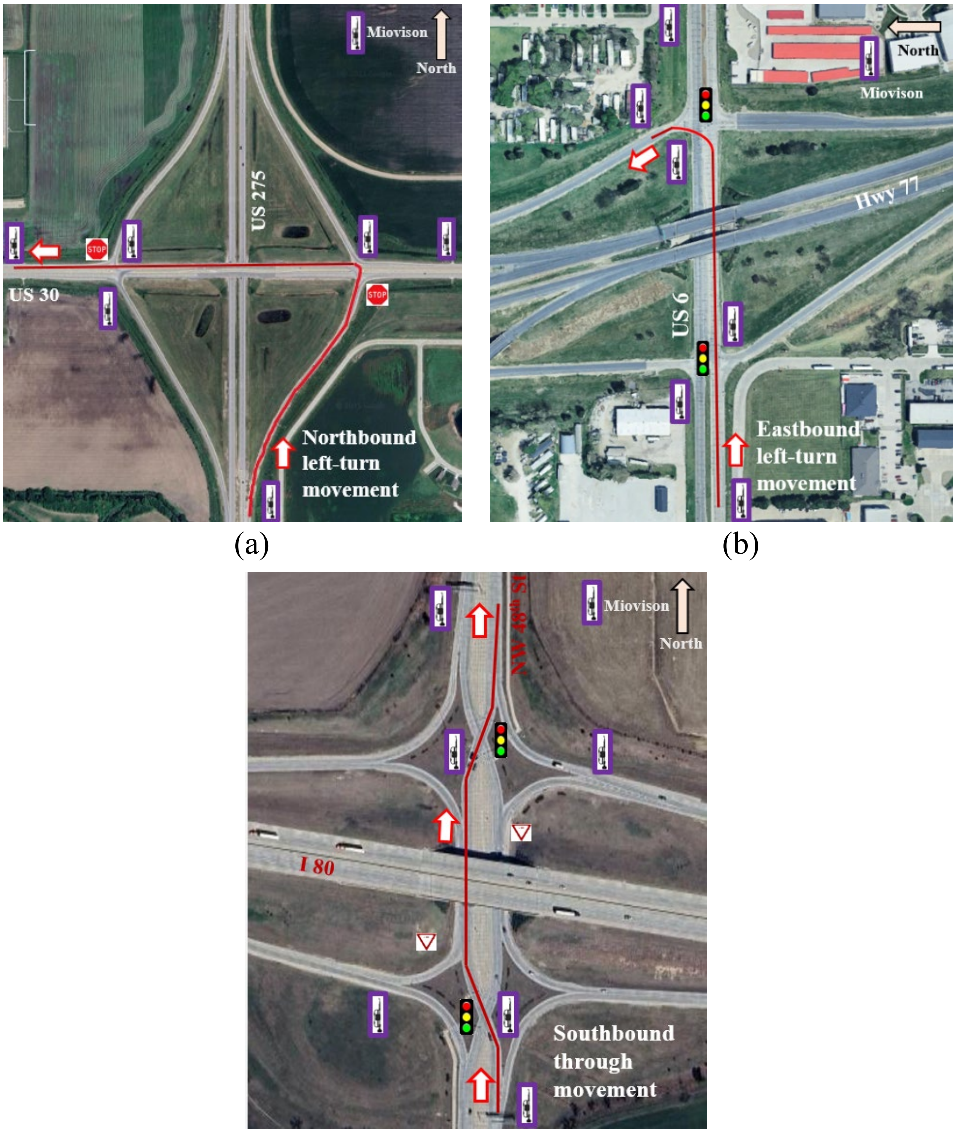
For gap acceptance behavior, critical headway helps to accurately model traffic behavior at the ramp intersection (HCM, 2022). VISSIM regulates these headway aspects using “priority rules” and “conflict area” functionalities (PTV VISSIM, 2024). The “minimum gap time” parameter under “priority rules” is tied to the critical headway.

This study selected parameters from car-following models and the minimum gap time to be calibrated for passenger cars and trucks, ensuring that the simulated interchange performance measure (i.e., travel time) matched the field-observed data. This calibration process employed statistical tests (Kolmogorov–Smirnov [KS] test) to confirm that the simulated and field-observed performance were statistically similar (see Step 2.3 in Figure 3.3). This methodology also conforms to the FHWA microsimulation guidelines (FHWA. Traffic Analysis Toolbox Volume III, 2019). Furthermore, this research used genetic algorithms (Kochenderfer, 2019) to find parameter values that yield statistically similar simulated and field-observed outcomes.

For Step 3, if multiple parameter outcomes satisfy the KS test, the research team selected parameter values with the lowest simulated-observed errors for the interchange. In addition, video data for headway and minimum gap time guided the selection of the final parameters listed in Table 4.1.

4.1.4 Microsimulation Model Calibration Results

The calibration process was performed by the algorithm shown in Figure 3.3. The process utilized the travel time distributions of the northbound left-turn movement at DIstop, the eastbound left-turn movement at DIsig, and the southbound through movement at DDI, respectively, as shown in Figure 4.2. The most critical movement, the left turn for DIsig, was chosen because traffic from the off-ramp must stop at the intersection and provide right-of-way to cross-street traffic. On the other hand, for DIsig, the left-turning traffic from the cross-street to the on-ramp had to cross the first signal and then execute the left turn through the second signal at the ramp terminals. For DDI, the through movement was selected because it must pass through both signals at the ramp terminals. Note that the length of these movements was around 3,300, 1,300, and 1,470 feet, respectively for DIstop, DIsig, and DDI.



(c)

Figure 4.2 Test sites used for model calibration (a) site 10: northbound left-turn movement at DIstop at Fremont, (b) site 11: eastbound left-turn at DIsig at Lincoln , (c) site 12: southbound through movement at DDI at Lincoln

Table 4.1 presents the final parameter values from peak hour periods and the corresponding operational results.

Table 4.1 Calibrated simulation parameter values for interchanges

Calibrated Simulation Parameters Values								
		DIstop			**DIsig**		**DDI**	
	Parameters	**PC**	**T**	**Parameters**	**PC**	**T**	**PC**	**T**
Car-following parameters	CC0 (ft)	10.0	12.0	AX	7.5	8.5	7.5	9.0
	CC1 (s)	*1.5 (0.2)	2.1 (0.4)	BX_add.	2.75	3.75	2.25	3.25
	CC2 (ft)	15.0	20.0	BX_mult.	4.25	4.75	4.25	4.75
	CC3-CC9	default	default					
Minimum gap acceptance (s)	Permitted left-turn from cross-street	-	-		5.1	6.8	-	-
	Left-turn from cross-street	5.0	6.9		-	-	-	-
	Left-turn from off-ramp	6.5	7.7		-	-	5.3	7.0
	Right-turn from off-ramp	6.0	7.3				6.2	7.7
	Permitted right - turn from off-ramp	-	-		6.7	7.5	-	-

Table 4.2 Performance comparisons of interchanges using calibrated parameters

Performance Comparisons Using Calibrated Parameters							
		DIstop		**DIsig**		**DDI**	
	Parameters	**Emp.**	**Sim.**	**Emp.**	**Sim.**	**Emp.**	**Sim.**
Travel time statistics	Average travel time (s)	143.6	141.4	60.1	62.5	64.5	62.9
	Mean absolute error (%)	1.5		4.2		2.4	
	Welch t-test (p-value)	0.89		0.44		0.69	
	KS-test (p-value)	0.19		0.09		0.18	

Table 4.2 shows the field-observed average travel times for DIstop, DIsig, and DDI for the selected movements, which were 143.6, 60.1, and 64.5 seconds, respectively. The calibrated simulation models produced travel times of 141.4, 62.5, and 62.9 seconds, corresponding to differences of 1.5%, 4.2%, and 2.4% from sites. Therefore, the simulated travel times closely aligned with field observations. Both the Welch t-test and the KS test yielded p-values exceeding 0.05, indicating there is no statistically significant difference between the simulated and

empirical values at a 95% confidence level. Furthermore, the KS test indicated that the distributions of simulated and observed travel times were statistically similar. Therefore, the simulation model replicated real-world variations in traffic performance.

4.1.5 Sensitivity Analysis Framework

This research conducted a comprehensive sensitivity analysis for three interchange types using the calibrated microsimulation model discussed above. For each interchange, the following combination of 2,048 traffic scenarios was simulated by varying:

- Cross-street AADT over eight levels (1,500; 2,500; 5,000; 10,000; 15,000; 20,000; 25,000; 30,000),
- Off-ramp AADT over four levels (500; 2,500; 5,000; 7,500),
- Turning movement distributions (left/through/right), in percentages of 5/90/5, 20/70/10, 40/40/20, and 60/10/30 for cross-street, and 5/0/95, 20/0/80, 40/0/60, and 60/0/40 for ramp (assuming no off-ramp traffic moved into the on-ramp).
- Time-of-day (TOD) for four periods:
    - Peak: two two-hour windows (e.g., 7–9 am and 4–6 pm),
    - Mid-day peak (e.g., 11am-1pm),
    - Off-peak daytime (Off-Peak 1): all remaining hours between 6 am-midnight, and
    - Off-peak nighttime (Off-Peak 2): midnight–6 am.

Based on observed volume patterns at three sites and NDOT's "Annual Traffic Count Data" report (NDOT ATC, 2025), each TOD period was assigned a representative AADT proportion. These detailed traffic periods were essential both for performance comparisons and subsequent benefit-cost analysis. The signal timings of three interchanges were optimized using Synchro for different TODs and traffic conditions.

For each of the three interchange types, 2,048 traffic scenarios (totaling 6,144) were simulated 10 times with different random seeds and appropriate warm-up periods, yielding a total of 61,440 simulation runs. Each run produced stopped delay and vehicular delay for left-turn, through (when applicable), and right-turn movements from four directions. Then delays were aggregated for the cross-street and ramp approaches and for the overall interchange. This process yielded 34 delay datapoints per run, totaling around two million. The entire workflow of simulation execution, data collection, and processing utilized a combination of full and partial automation.

For a fair comparison, all interchanges' lane distributions were kept the same, except for the fundamental geometrical differences among them. In this study, a single off-ramp (50 mph speed limit) meets four-lane cross-streets (45 mph speed limit near the ramp intersection). In each direction, the cross streets have two through lanes. Cross-streets from DIstop and DIsig have one dedicated right-turn lane before approaching ramp intersections. At two ramp intersections, one left-turn storage lane is provided in each direction for traffic approaching the on-ramp. For DDI, within two ramp intersections, there is one left-turn lane in each direction for traffic moving to on-ramp. The traffic stream consists of 10% trucks at all interchanges.

This sensitivity framework supports the interchange comparison and selection models presented in the rest of the chapter.

4.2 Performance Evaluation and Predictive Modeling

4.2.1 Comparative Performance Evaluation

A subset of sensitivity results is shown in Figures 4.3, 4.4, and 4.5 representing the peak hour, all AADT combinations, and varying left-turn percentages (four and two levels for cross-streets and ramp approaches) for DIstop, DIsig, and DDI, respectively. The average interchange delays are listed as HCM-defined LOS (HCM, 2022) for three interchange types. Appendix B includes the LOS thresholds.

			Ramp AADT							
			500		2500		5000		7500	
LOS Lists		Left Turn %	5	60	5	60	5	60	5	60
Cross-street AADT	1500	5	A	A	A	A	A	A	A	B
		20	A	A	A	A	A	A	A	B
		40	A	A	A	A	A	A	A	B
		60	A	A	A	A	A	A	A	B
	2500	5	A	A	A	A	A	A	A	B
		20	A	A	A	A	A	A	A	B
		40	A	A	A	A	A	A	A	B
		60	A	A	A	A	A	A	A	B
	5000	5	A	A	A	A	A	A	A	B
		20	A	A	A	A	A	A	A	B
		40	A	A	A	A	A	A	A	B
		60	A	A	A	A	A	A	A	B
	10000	5	A	A	A	A	A	B	B	F
		20	A	A	A	A	A	B	A	F
		40	A	A	A	A	A	B	A	F
		60	A	A	A	A	A	B	A	F
	15000	5	A	A	A	A	A	D	B	F
		20	A	A	A	A	A	E	B	F
		40	A	A	A	B	A	F	B	F
		60	A	B	B	D	B	F	C	F
	20000	5	A	A	A	B	A	F	C	F
		20	A	A	A	C	A	F	C	F
		40	B	C	B	D	C	F	D	E
		60	F	F	F	F	F	F	F	F
	25000	5	A	A	A	C	B	F	F	F
		20	A	B	A	D	B	F	F	F
		40	F	F	F	F	F	F	F	F
		60	F	F	F	F	F	F	F	F
	30000	5	A	A	A	C	C	F	F	F
		20	B	B	B	D	C	F	F	F
		40	F	F	F	F	F	F	F	F
		60	F	F	F	F	F	F	F	F

Figure 4.3 LOS of selected scenarios for stop-controlled diamond interchange for peak hour periods

		Ramp AADT							
		500		2500		5000		7500	
LOS Lists	Left Turn %	5	60	5	60	5	60	5	60
Cross-street AADT 1500	5	A	B	A	B	B	C	B	C
	20	A	B	A	B	B	C	B	C
	40	A	B	A	B	B	C	B	C
	60	A	B	A	B	B	C	B	C
2500	5	B	B	B	B	B	C	B	C
	20	B	B	B	B	B	C	B	C
	40	B	B	B	B	B	C	B	C
	60	B	B	B	B	B	C	B	C
5000	5	B	B	B	B	B	C	B	C
	20	B	B	B	B	B	C	B	C
	40	B	B	B	B	B	C	B	C
	60	B	B	B	B	B	C	B	C
10000	5	B	B	B	B	B	C	B	C
	20	B	B	B	B	B	C	B	C
	40	B	B	B	B	B	C	B	C
	60	B	B	B	B	B	C	B	C
15000	5	B	C	B	C	B	C	B	C
	20	B	C	B	C	B	C	B	C
	40	B	C	B	C	B	C	B	C
	60	C	C	C	C	C	C	C	C
20000	5	C	C	C	C	C	C	C	C
	20	C	C	C	C	C	C	C	C
	40	C	C	C	C	C	C	C	C
	60	D	D	D	D	D	D	D	D
25000	5	C	C	C	C	C	C	C	C
	20	C	C	C	C	C	C	C	C
	40	C	C	C	C	C	C	C	C
	60	F	F	F	F	F	F	F	F
30000	5	D	D	D	D	D	D	D	D
	20	D	D	D	D	D	D	D	D
	40	D	D	D	D	D	D	D	D
	60	F	F	F	F	F	F	F	F

Figure 4.4 LOS of selected scenarios for signalized diamond interchange for peak hour periods

		Ramp AADT							
		500		2500		5000		7500	
LOS Lists	Left Turn %	5	60	5	60	5	60	5	60
Cross-street AADT 1500	5	A	B	A	B	A	B	A	B
	20	A	B	A	B	A	B	A	B
	40	A	B	A	A	A	B	A	B
	60	A	A	A	A	A	A	A	B
2500	5	B	B	A	B	A	B	A	B
	20	B	B	A	B	A	B	A	B
	40	A	B	A	B	A	B	A	B
	60	A	A	A	A	A	A	A	B
5000	5	B	B	B	B	B	B	B	B
	20	B	B	B	B	A	B	A	B
	40	B	B	A	B	A	B	A	B
	60	A	A	A	A	A	B	A	B
10000	5	B	B	B	B	B	B	B	B
	20	B	B	B	B	B	B	B	B
	40	B	B	B	B	B	B	B	B
	60	B	B	A	B	A	B	A	B
15000	5	B	B	B	B	B	B	B	B
	20	B	B	B	B	B	B	B	B
	40	B	B	B	B	B	B	B	B
	60	B	B	B	B	B	B	B	B
20000	5	C	C	C	C	B	C	B	C
	20	B	B	B	B	B	B	B	B
	40	B	B	B	B	B	B	B	B
	60	B	B	B	B	B	B	B	B
25000	5	C	C	C	C	C	C	C	C
	20	C	C	C	C	C	C	C	C
	40	C	C	C	C	C	C	C	C
	60	B	B	B	B	B	B	B	B
30000	5	C	C	C	C	C	C	C	C
	20	C	C	C	C	C	C	C	C
	40	C	C	C	C	C	C	C	C
	60	F	F	E	E	E	E	E	E

Figure 4.5 LOS of selected scenarios for the diverging diamond interchange for peak hour periods

Figure 4.3 shows that up to 5,000 AADT for both approaches, LOS of DIstop remained within the A-B range. For higher left-turn percentage and higher ramp-AADT, the LOS deteriorated after 10,000 cross-street-AADT to C-F. These trends met expectations. As the cross-street AADT increased, vehicles from off-ramps suffered higher delays due to fewer available gaps for merging or crossing maneuvers.

Figure 4.4 shows that DIsig performed better by maintaining LOS A-C up to 15,000 cross-street AADT. On and beyond, 20,000 cross-street-AADT, DIsig's LOS worsened as left-turn traffic from cross-streets increased. This illustrates the critical operational aspects of left-turn traffic for high-volume cross-street traffic at an interchange.

On the other hand, DDI in Figure 4.5, as expected, handled the high left-turn traffic more efficiently. For cross-street-AADT up to 15,000, DDI maintained LOS A-B. In a few scenarios with a cross-street AADT of 30,000, LOS worsened to E-F. For higher cross-street-AADT of 20,000 and 25,000, it reached LOS C at most. For 30,000 cross-street AADT, a few scenarios resulted in LOS E-F. Note that in many DDI scenarios, increasing the cross-street left-turn share from 5% to 60% reduced through traffic from 90% to 10%, thereby improving LOS. This improvement is attributed to fewer vehicles needing to pass through the second ramp terminal signal, thereby reducing delay and improving overall LOS.

Note that the LOS concept is based on drivers' experiences of interchange operations. While non-peak-hour traffic demand is lower, traffic agencies often tolerate LOS F during peak hours if the queue does not block nearby intersections or queue spillback does not occur frequently. Therefore, interchange conversion decisions are not based solely on LOS but on a comprehensive assessment of delay magnitude, TOD conditions, safety, and costs. Nevertheless, LOS is a key aspect in intersection preference decisions.

4.2.2 Predictive Performance Model

A series of log-linear regression models was developed to quantify and understand the factors influencing interchange delays across thousands of scenarios. Equation (4.1) shows the general form of the model.

$$\log(D) = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \ldots\ldots + \beta_k X_k \tag{4.1}$$

Here, D is the estimated average delay (interchange, cross-street, or ramp) in sec/veh. β_0 is the intercept and $\beta_1, \beta_2, \ldots, \beta_k$ are the estimated coefficients of the predictors $X_1, X_2, \ldots, X_k$. The predictors are cross-street AADT (AADT_C) and off-ramp AADT (AADT_R) in thousands, left-turn percentage from cross-street (LT_C) and off-ramp (LT_R), and TODs for peak hour (TOD_PEAK), midday peak (TOD_MID), off-peak hour 1 (TOD_OFF1), and off-peak hour 2 (TOD_OFF2).

Note that the through (when applicable) and right-turn percentages are excluded from the model to prevent multicollinearity, as they are directly associated with the left-turn percentage, which sums to 100%.

The log-linear model was applied because the corresponding delays were right-skewed. Higher traffic demand or left-turns cause substantial delay compared to their lower-delay counterparts. Additionally, log-linear models do not predict negative delays, unlike linear regression, and provide a better fit.

Tables 4.3 and 4.4 show model outcomes for peak, midday peak, and all TODs, including estimated (Est.) coefficients, standard errors (SE), p-values (p val.), and corresponding model fit for all interchanges. Note that the coefficient for TOD_PEAK was set as zero as it is redundant in the model. Table 4.3 includes intersection delays for three interchange types, and Table 4.4 shows cross-street and ramp approach delays for DDI.

Table 4.3 Log-linear regression models for average delays (peak, mid-day, all TODs)

	Parameters	Peak Hours			Mid-Day Peak Hours			All TODs		
		Est.	**SE**	**p val.**	**Est.**	**SE**	**p val.**	**Est.**	**SE**	**p val.**
DIstop	Intercept	-1.041	0.111	0.000	-0.873	0.102	0.000	0.504	0.069	0.000
	AADT_C	0.138	0.003	0.000	0.101	0.003	0.000	0.068	0.001	0.000
	AADT_R	0.268	0.013	0.000	0.220	0.012	0.000	0.227	0.007	0.000
	LT_C	0.030	0.001	0.000	0.025	0.001	0.000	0.019	0.0009	0.000
	LT_R	0.010	0.001	0.000	0.009	0.001	0.000	0.006	0.0009	0.000
	TOD_PEAK	-	-	-	-	-	-	0	-	-
	TOD_MID	-	-	-	-	-	-	-0.703	0.055	0.000
	TOD_OFF1	-	-	-	-	-	-	-1.418	0.055	0.000
	TOD_OFF2	-	-	-	-	-	-	-2.770	0.055	0.000
	Adjusted R^2	*0.811*			*0.750*			*0.726*		
DIsig	Intercept	2.479	0.051	0.000	2.556	0.025	0.000	2.871	0.025	0.000
	AADT_C	0.047	0.001	0.000	0.027	0.0008	0.000	0.021	0.0007	0.000
	AADT_R	0.013	0.006	0.037	0.012	0.003	0.000	0.030	0.002	0.000
	LT_C	0.007	0.0007	0.000	0.003	0.0003	0.000	0.003	0.0003	0.000
	LT_R	0.004	0.0007	0.000	0.004	0.0003	0.000	0.006	0.0003	0.000
	TOD_PEAK	-	-	-	-	-	-	0	-	-
	TOD_MID	-	-	-	-	-	-	-0.334	0.020	0.000
	TOD_OFF1	-	-	-	-	-	-	-0.550	0.020	0.000
	TOD_OFF2	-	-	-	-	-	-	-1.614	0.020	0.000
	Adjusted R^2	*0.652*			*0.725*			*0.800*		
DDI	Intercept	2.565	0.030	0.000	2.602	0.014	0.000	2.945	0.017	0.000
	AADT_C	0.043	0.0009	0.000	0.031	0.0004	0.000	0.031	0.0004	0.000
	AADT_R	-0.015	0.003	0.000	-0.018	0.001	0.000	-0.026	0.001	0.000
	LT_C	-0.002	0.0004	0.000	-0.006	0.0002	0.000	-0.008	0.0002	0.000
	LT_R	0.003	0.0004	0.000	0.003	0.0002	0.000	0.003	0.0002	0.000
	TOD_PEAK	-	-	-	-	-	-	0	-	-
	TOD_MID	-	-	-	-	-	-	-0.234	0.013	0.000
	TOD_OFF1	-	-	-	-	-	-	-0.415	0.013	0.000
	TOD_OFF2	-	-	-	-	-	-	-0.480	0.013	0.000
	Adjusted R^2	*0.876*			*0.827*			*0.814*		

Table 4.4 Log-linear regression models for cross-street and ramp delays for DDI (peak, mid-day, all TODs)

	Parameters	Peak Hours			Mid-Day Peak Hours			All TODs		
		Est.	**SE**	**p val.**	**Est.**	**SE**	**p val.**	**Est.**	**SE**	**p val.**
DDI (cross-street delay)	Intercept	2.884	0.032	0.000	2.945	0.009	0.000	3.292	0.168	0.000
	AADT_C	0.035	0.001	0.000	0.022	0.0002	0.000	0.020	0.0004	0.000
	AADT_R	0.0001	0.003	0.967	0.0001	0.001	0.929	0.00008	0.001	0.961
	LT_C	-0.004	0.0005	0.000	-0.008	0.0001	0.000	-0.010	0.0003	0.000
	LT_R	0.00001	0.0005	0.968	0.00005	0.0001	0.967	0.00009	0.0002	0.968
	TOD_PEAK	-	-	-	-	-	-	0	-	-
	TOD_MID	-	-	-	-	-	-	-0.226	0.013	0.000
	TOD_OFF1	-	-	-	-	-	-	-3.941	0.013	0.000
	TOD_OFF2	-	-	-	-	-	-	-0.357	0.013	0.000
	Adjusted R^2	*0.705*			*0.947*			*0.713*		
DDI (ramp delay)	Intercept	1.391	0.027	0.000	1.330	0.029	0.000	1.292	0.029	0.000
	AADT_C	0.029	0.0008	0.000	0.021	0.0009	0.000	0.018	0.0008	0.000
	AADT_R	0.082	0.003	0.000	0.060	0.003	0.000	0.105	0.003	0.000
	LT_C	-0.005	0.0004	0.000	-0.003	0.0004	0.000	-0.004	0.0003	0.000
	LT_R	0.017	0.0004	0.000	0.018	0.0004	0.000	0.021	0.0003	0.000
	TOD_PEAK	-	-	-	-	-	-	0	-	-
	TOD_MID	-	-	-	-	-	-	-0.192	0.232	0.000
	TOD_OFF1	-	-	-	-	-	-	-0.442	0.232	0.000
	TOD_OFF2	-	-	-	-	-	-	-1.393	0.232	0.000

Table 4.3 offers quick references for practitioners to delay predictions for thousands of scenarios. For example, for all TODs, DIsig's coefficients for AADT_C, AADT_R, LT_C, and LT_R are positive, meaning that increasing these factors contributes to higher delays. To be precise, every unit (1,000 AADT) increase of cross-street-AADT, provided other variables are constant, increased the intersection delay by 2.1% (coefficient =0.021). Similarly, a 1% increase in ramp-AADT and a 1% increase in cross-street and ramp left-turning percentage resulted in 3.0%, 0.3%, and 0.6% higher delays, respectively. On the other hand, TOD effects relative to peak hour showed considerably lower delays during midday, daytime off-peak, and nighttime off-peak, by around 28%, 42%, and 80%, respectively. Note that all these estimated coefficients (β) values were statistically significant as listed by their corresponding p-values.

Using the results from Table 4.3, Equation (4.2) illustrates an example application of a log-linear regression model for average intersection delays for DIsig across different TODs.

$$\log(D_{\text{DIsig.int.delay}}) = 2.871 + (0.021\times AADT\_C) + (0.030\times AADT\_R) + (0.003\times LT\_C) + (0.006\times LT\_R) + (0\times TOD\_PEAK) - (0.334*TOD\_MID) - (0.550*TOD\_OFF1) - (1.614*TOD\_OFF2) \quad (4.2)$$

Where,

$D_{\text{DIsig.int.delay}}$ = average intersection delay for DDI

AADT_C = cross-street-AADT (in thousands); application range 1,500-30,000

AADT_R = ramp-AADT (in thousands); application range 500-7,500

LT_C = cross-street left-turning traffic percentage; application range 5%-60%

LT_R = ramp left-turning traffic percentage; application range 5%-60%

TOD_PEAK = 0 (other TOD coefficients are estimated relative to the peak period)

TOD_MID = 1 for midday period; 0 otherwise

TOD_OFF1 = 1 for off peak 1 period; 0 otherwise

TOD_OFF2 = 1 for off peak 2 period; 0 otherwise

Furthermore, Table 4.3 shows that, unlike DIstop and DIsig, the estimated coefficients for AADT_R and LT_C are negative. As already discussed, increasing left-turn traffic from the cross-street is related to reducing the through traffic to the second ramp signal, causing lesser delays; hence, the observed negative coefficient for LT_C. However, a negative coefficient for ramp traffic may seem counterintuitive. Therefore, Table 4.4 includes cross-street and ramp delays for DDI and shows that both AADT_R coefficients are positive. It was observed that, because intersection delays are calculated as a weighted average across all approaches, the lower delay that ramp traffic experiences contributes to overall delay reductions at interchanges as ramp traffic increases. Note that this observation is limited to the AADT ranges tested and the yield-controlled ramp used for DDI in this study.

Chapter 5 Safety Studies of Unsignalized Intersections and Interchanges

5.1 Safety Studies of Intersections

Intersections with stop control on the minor approach, such as TWSC intersections, represent the predominant form of at-grade access along Nebraska's rural expressway system. While these intersections serve an essential function in maintaining mobility and access across high-speed divided corridors, they are frequently associated with severe crashes resulting from the combination of high approach speeds, limited sight distances, and complex crossing or turning maneuvers required of minor-road drivers (Early et al., 2016). These operational and safety challenges make TWSC intersections a critical focus area in improving the safety performance of rural expressways.

The objective of this section was to evaluate the safety characteristics and crash patterns of rural TWSC intersections across Nebraska using available traffic, geometric, and crash data. The results from this assessment established the baseline safety performance for existing TWSC intersections, which will subsequently be used as the reference condition for quantifying the safety benefits of converting TWSC intersections to alternative designs such as multi-lane roundabouts and RCUT intersections through the application of appropriate CMFs. A CMF is a multiplicative factor used to estimate the expected change in crash frequency after implementing a specific roadway treatment or design change. In the context of this project, a CMF represents the change in crashes expected when converting from one intersection type to another.

5.1.1 Safety Studies of Minor Approach Stop-Controlled Intersections

NDOT previously conducted a comprehensive safety and operational study of rural intersections with stop-controlled minor approaches to evaluate the influence of FRT ramps on intersection performance and crash occurrence (Khattak, Camenzind, and Haque, 2023). That investigation examined 68 rural intersections with FRT ramps and 24 comparable rural intersections without FRT ramps to determine whether FRT ramps were associated with measurable differences in crash frequency, crash rate, or crash severity. The analysis utilized NDOT's statewide crash database covering the 2010–2019 period and employed statistical tests to assess the significance of observed differences between the two intersection groups. The results indicated that the differences were not statistically significant, suggesting that the presence of FRT ramps did not alter the overall crash experience at stop-controlled rural intersections.

While earlier Nebraska intersection-based research focused mainly on two-lane rural highways, the current project expanded the scope to include TWSC intersections on rural expressways, which are four-lane highways with partial access control. This project extended the database of FRT and non-FRT intersections to encompass a broader range of geometric and traffic conditions representative of Nebraska's expressway system.

5.1.1.1 Data Collection and Processing

The research team had access to the detailed intersection database and all relevant research materials developed under the FRT Project. Out of the original 92 rural intersections (68 with FRT ramps and 24 without FRT ramps), 86 intersections were retained for this study. The remaining sites were excluded due to specific data or geometric considerations, including intersections located very close to one another and sites that had recently undergone geometric modifications. For example, the intersection at US-75 and Highway 1 near Murray was removed from the analysis because it was recently reconstructed as an RCUT intersection. Out of the selected 86 intersections, six were located on four-lane highways.

In addition to the existing intersection database, the research team collected detailed geometric, traffic, and crash data for TWSC intersections located on rural expressways across Nebraska. For each intersection, a comprehensive set of geometric and operational characteristics was compiled, including geographic coordinates, major and minor roadway names and route numbers, presence of FRT ramps, median type, intersection angle, control type, number of legs, pavement surface type, number and type of lanes, presence of lighting, and posted speed limits for each approach. Most of these intersections are located along US-81 (from Chester to York, south of Columbus, and from Madison to Norfolk), US-75 (from Cortland to Hickman and from west of Waverly to Wahoo), and US-275 near West Point. The selected intersection sites met specific criteria, including a rural setting, paved minor approaches, and the availability of AADT data for both major and minor road approaches. A complete list of the newly selected 22 rural expressway intersections is provided in Appendix C. Figure 5.1 presents the geographic distribution of all 28 intersections located on four-lane expressways and 80 intersections located on two-lane rural highways included in this study across the Nebraska state highway system.

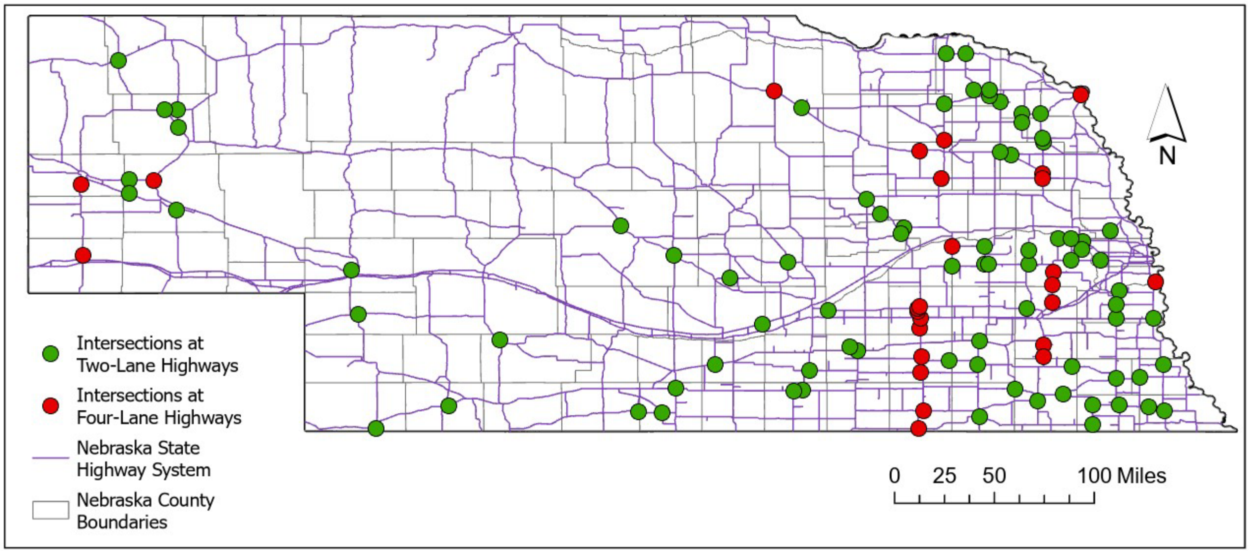


Figure 5.1 Selected rural intersections in the Nebraska State Highway System

In general, AADT on rural highways in Nebraska fluctuates with relatively small magnitudes from year to year. For the 86 intersections selected from the FRT project, AADT values from 2018 were projected through 2024 using an annual AADT growth factor derived from data collected at 12 automatic traffic recorder (ATR) stations across Nebraska, in proximity to these study sites. This factor was developed based on observed AADT trends between 2018 and 2024. Then, the projected AADT data from 2020 to 2024 were used for the analysis. For the 22 intersections located on rural expressways, AADT data for 2020 through 2024 were obtained directly from NDOT's AADT count database on the NDOT ArcGIS online mapping platform.

The NDOT provided crash data for Nebraska from 2020 to 2024, along with the crash location geographic coordinates (latitude and longitude). These crashes were uploaded to ArcGIS and plotted using their geographic coordinates. Then, shapefiles for the intersections were created and plotted along with the crashes in ArcGIS. This research considered crashes reported within a quarter-mile of the intersection's center point, similar to the FRT project's research approach. Therefore, for each intersection, polygon buffers were created in ArcGIS with a radius of 0.25 miles, and crashes occurring within these buffers were then exported into separate shapefiles for each intersection (Khattak et al., 2024; Haque et al., 2024). Figure 5.2 illustrates this process for the TWSC intersection at the junction of Hwy 81 and US 136 at Hebron, Nebraska. The selected 108 intersections were mapped in this way.

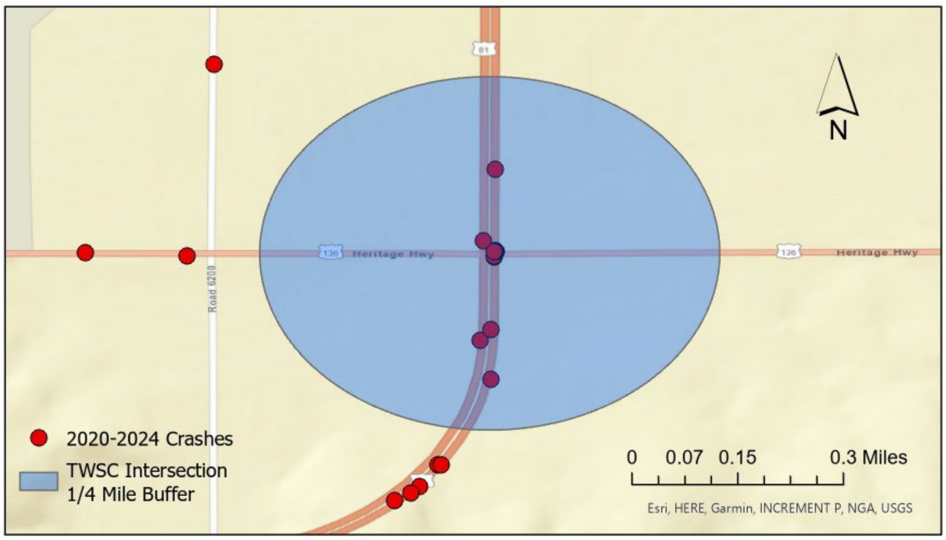


Figure 5.2 Crashes from 2020-2024 at US-81/US-136 TWSC intersection

Utilizing crash data for each intersection, Figure 5.3 compares the crash severity experienced at the TWSC intersections analyzed. These categories were presented on the x-axis according to the KABC crash reporting procedure: fatal injury (K), suspected serious injury (A), suspected minor injury (B), and possible injury (C). Following this, the non-injury categories, property damage only (PDO) and non-reportable, were plotted. Overall, the analysis revealed that 2.1% of the total crashes were fatal, and non-fatal injuries accounted for around 34.4%. PDO was the most common type of crash, accounting for around 54%.

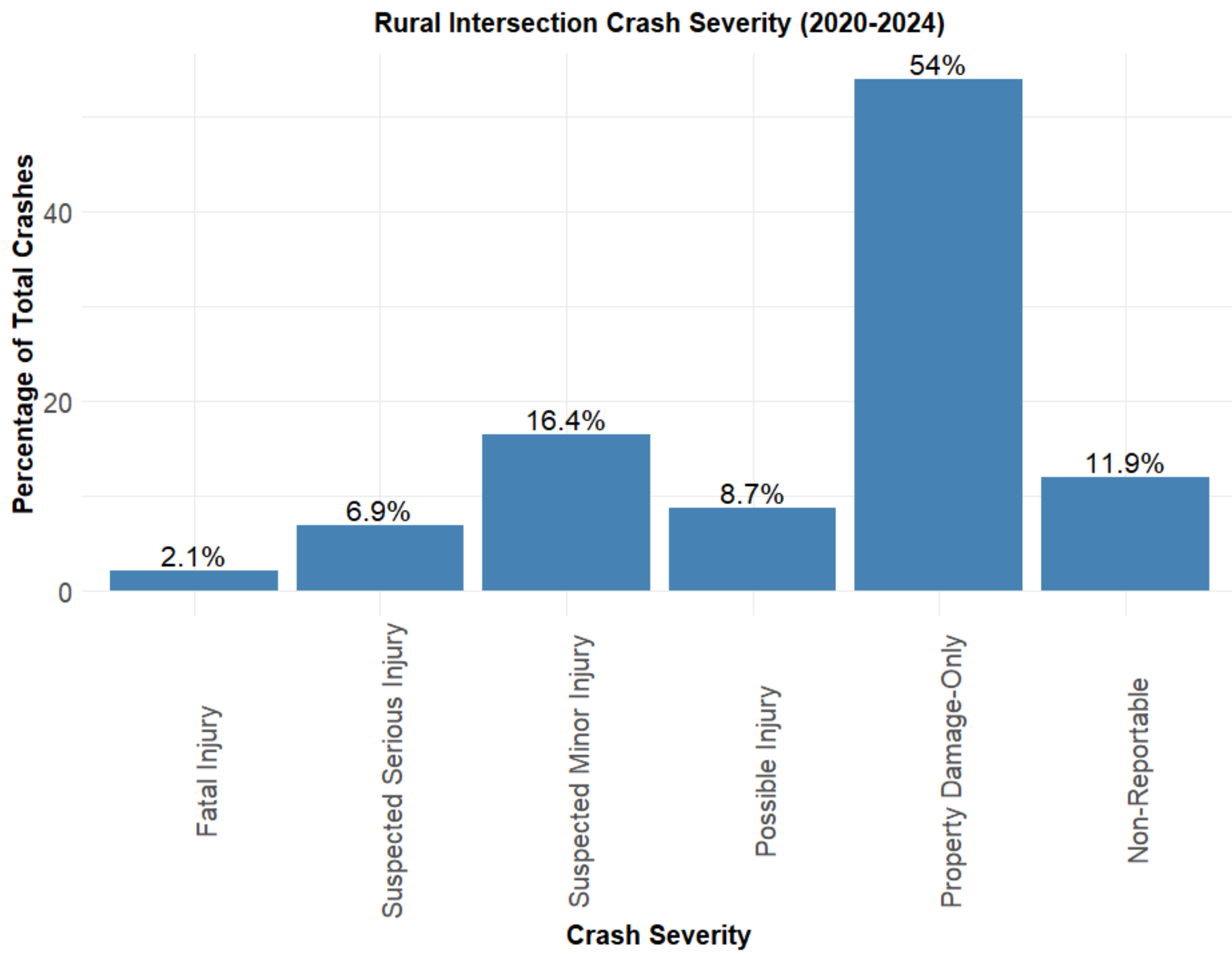


Figure 5.3 Crash severity of TWSC intersections

Figure 5.4 illustrates the distribution of crash types observed at TWSC intersections along rural expressways. Among all recorded crashes, angle collisions accounted for the largest share (47.5%), indicating their predominance at these intersection types. Non-motor crashes, defined as crashes involving a motor vehicle and a non-motorized entity, accounted for 32.5% of total crashes. Note that there were no Rear-to-Rear or Rear-to-Side types of crashes reported.

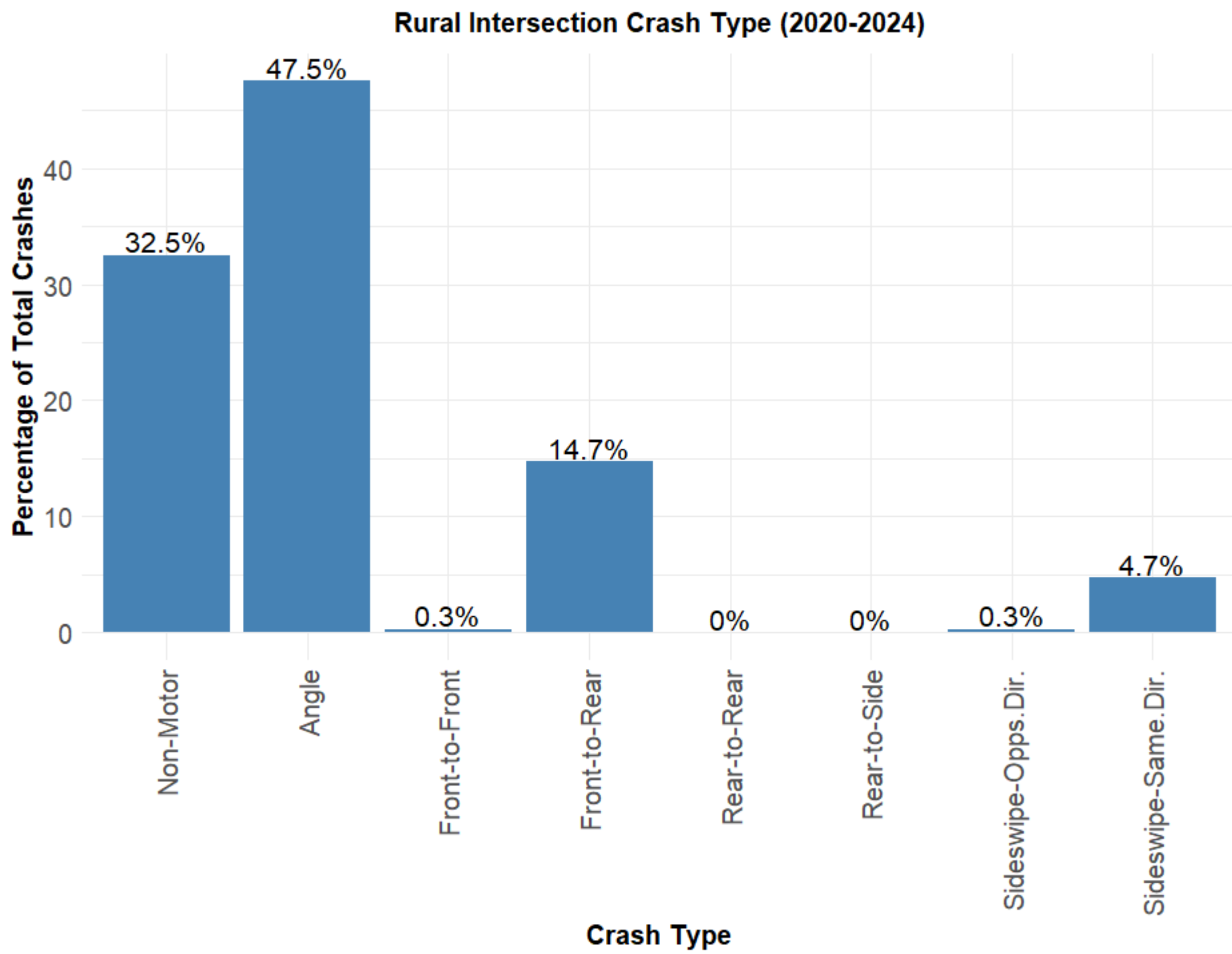


Figure 5.4 Crash type of TWSC intersections

5.1.1.2 Research Method and Analysis

Crash frequency represents the safety for roadway segments and intersections. Crash frequency (F) is a straightforward crash count over a specified time period (usually 12 months) or the total number of crashes (C) divided by the number of years (N), as shown in Equation (5.1), yielding crashes per year as the output.

$$F = \frac{C}{N} \tag{5.1}$$

A limitation of relying on simple crash frequency for safety assessment is that it does not account for traffic volume, which may substantially impact crash frequency. Therefore, when comparing a low-AADT intersection to a high-AADT intersection, the latter will inherently have a higher crash frequency due to greater crash exposure (i.e., a higher likelihood of crashes).

Crash rate, on the other hand, accounts for exposure, setting all locations from low to high AADT on an even playing field. The crash rate (R) is calculated using Equation (5.2), with the total number of crashes in the study period (C), the number of years of data (N), and the daily entering traffic volume (V). Crash rates are generally given as crashes per million entering vehicles.

$$R = \frac{C * 1{,}000{,}000}{N * V * 365} \tag{5.2}$$

The impact of AADTs along with other traffic and geometric factors on crash counts can be obtained by crash frequency models as well. Such a model enables the estimation of expected crash frequencies for intersections operating under different AADT conditions, thereby allowing a consistent, data-driven comparison of safety performance across varying traffic and design scenarios.

Crash frequencies comprise of count data that are appropriately modeled with the Poisson family of models. The basic Poisson model is a statistical model used to analyze count data, assuming the counts follow a Poisson distribution, which describes the probability of observing a certain number of events in an interval or area. Therefore, the Poisson distribution is a probability distribution characterized by a single parameter, λ, which represents the mean or expected value of the count data. The distribution assumes that events occur independently and at a constant rate over time or area. The count data are modeled as a function of one or more explanatory or independent variables, which can be categorical or continuous. The model assumes that the logarithm of the expected count, denoted by log(λ), is a linear function of the explanatory variables. Equation (5.3) shows the model.

$$\log(\lambda) = \alpha + \beta_1 x_1 + \beta_2 x_2 + \ldots + \beta_p x_p \tag{5.3}$$

Where,

λ = mean or expected value of the count data,

x = the explanatory variable,

α = the intercept,

β = coefficients of the explanatory variables.

An advantage of the Poisson model is its simplicity and ease of interpretation. It assumes that the events occur independently and at a constant rate, which makes it suitable for analyzing count data that satisfies these assumptions. However, the model assumes that the variance of the count data equals its mean, which may not hold up in practice. When the variance of the count data exceeds the mean, indicating overdispersion, the negative binomial model may be more appropriate.

The negative binomial model is a statistical model for count data when the data exhibit overdispersion. The negative binomial distribution is a probability distribution that describes the probability of observing a certain number of events in a given interval or area. The distribution is characterized by two parameters: the mean or expected value, denoted by μ, and the dispersion parameter, denoted by α. The mean represents the average number of events expected to occur, and the dispersion parameter measures the degree of variation in the data. The count data are modeled as a function of one or more explanatory variables, which, as discussed earlier, can be categorical or continuous. The model assumes that the count data follow a negative binomial distribution and estimates its parameters using maximum likelihood. Equation (5.4) shows the model equation. The log link function in the equation ensures that the predicted values are non-negative.

$$\log(E(Y|x)) = \beta_o + \beta_1 x1 + \beta_2 x_2 + \ldots + \beta_p x_p \tag{5.4}$$

Where,

Y = the count data,

x = the explanatory variable,

β_o = the intercept,

$\beta_1, \beta_2, \ldots, \beta_p$ = the estimated coefficients of the explanatory variables.

One of the main advantages of the negative binomial model is its flexibility in handling overdispersed count data. The model can also handle data with excess zeros, which occur when a large proportion of the data points have a count of zero. This is achieved by adding an extra parameter to the model, known as the zero-inflation parameter, which measures the proportion of excess zeros in the data.

The 5-year crash frequency for each of the 108 intersections was computed using Equation (5.1). Preliminary data analysis indicated overdispersion, suggesting that a negative binomial regression model was the appropriate analytical approach. Accordingly, the five-year crash frequency data were modeled using a negative binomial framework to identify the factors influencing crash occurrence at rural intersections. A range of independent variables was examined for their potential effects on crash frequency, including the five-year total AADT (expressed in thousands of vehicles per day), highway type (two-lane or four-lane), number of intersection legs (three-leg or four-leg configurations), and intersection skew angle, among others.

Table 5.1 presents the estimated model results. The model consists of a constant, the five-year total AADT (in thousands), and a four-lane highway (i.e., expressway) indicator variable that distinguishes between intersections located on two-lane and four-lane highways. It also includes the dispersion parameter alpha. The statistical significance of this parameter indicates the appropriateness of the negative binomial model. Also, greater values of five-year total AADT were statistically significantly associated with greater five-year crash frequencies (i.e., crashes increase with increasing AADT). The four-lane highway indicator was statistically significant, showing a difference in five-year crash frequencies at four-lane and two-lane highway intersections after accounting for the AADT effect. Various other variables were tested in the model specification, but none showed statistical significance.

Table 5.1 Negative binomial model for 5-year crash frequency using 2020-2024 crashes

Parameter	**Description**	**Estimated Coefficient**	**Standard Error**	**Z-Statistic**	**P-Value**
Constant	Model constant	0.882	0.114	7.68	0.000
Total AADT	Total 5-year AADT in thousands	0.016	0.001	8.40	0.000
Four-Lane Highways Indicator	1 if four-lane highways, 0 for two-lane highways	0.748	0.111	6.73	0.000
Alpha	Dispersion parameter	0.135	0.041	3.28	0.001

Model Summary Statistics	
Number of observations	108
Log likelihood (LL) function	-277.937
Restricted LL function	-318.739
Chi-squared (P-value)	81.60519 (0.000)
R-squared (McFadden)	0.128

Similar to the comparative operational analyses presented in Chapters 3 and 4, a range of traffic conditions was examined to support the development of guidelines for evaluating alternative intersection designs. The analysis considers seven major-road AADT levels (1,500; 2,500; 5,000; 10,000; 15,000; 20,000; and 25,000 vehicles per day) and four minor-road AADT levels (500; 2,500; 5,000; and 7,500 vehicles per day). These combinations represent intersection AADT or entering AADT values (calculated as the sum of AADTs from all approaches or legs, divided by two for a four-legged intersection) ranging approximately from 2,000 to 32,500 vehicles per day.

The negative binomial crash frequency model for TWSC intersections on rural four-lane highways (as shown in Table 5.1) was developed using five-year AADT totals ranging from 24,096 to 84,040, corresponding to per-year AADT values of approximately 4,819 to 16,808 vehicles per day. Therefore, it is recommended that the negative binomial model be applied only within this AADT range to ensure reliable crash frequency prediction. Application of the model outside this AADT bound may yield inaccurate or biased crash frequency estimates. Therefore, crash frequencies for intersections outside of the AADT range can be estimated using the procedures recommended in the Highway Safety Manual (HSM, 2010).

The Highway Safety Manual (2010) provides a predictive model for estimating average crash frequency at rural intersections on multilane highways. The effect of traffic volume (i.e., AADT) on crash frequency is incorporated through the safety performance functions (SPFs). The effects of geometric design and traffic control features are incorporated through the crash modification factors (CMFs). The SPF developed for the TWSC intersection is applicable to both divided and undivided rural four-lane highways. Equation (5.5) lists the SPF for TWSC intersections for the base condition, total crashes (i.e., all severity levels of crashes and non-injury crashes included) (HSM, 2010).

$$N_{spf\ int} = \exp\left[a + b \times \text{In}\ (AADT_{maj}) + c \times \text{In}\ (AADT_{min})\right] \tag{5.5}$$

Where,

$N_{spf\ int}$ = 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,

a, b, c = regression coefficients (the values of a, b, and c are -10.008, 0.848, and 0.448, respectively, for four-legged intersection for total crashes).

Equation (5.5) is applicable for $AADT_{maj}$ ranging from 0 to 78,300 vehicles per day and $AADT_{min}$ ranging from 0 to 7,400 vehicles per day, and the following base conditions:

- Intersection skew angle is 0°,
- The number of intersection left-turn lane is 0, except on stop-controlled approaches,
- The number of intersections right-turn lane is 0, except on stop-controlled approaches, and
- Intersection is not lighted.

The CMF for total crashes (HSM, 2010) for intersection angle (four-legged) is presented in Equation (5.6).

$$CMF_{1i} = \frac{0.053 \times skew}{(1.43+0.53\times skew)} + 1.0 \tag{5.6}$$

Where,

CMF_{1i} = crash modification factor for the effect of intersection skew on total crashes,

skew = intersection skew angle (in degrees); the absolute value of the difference between 90 degrees and the actual intersection angle.

The skew angle is considered 0 degrees for this project rendering a CMF_{1i} value 1.0.

On the other hand, the CMF for a lighted intersection (HSM 2010; Elvik and Vaa, 2004) is expressed by Equation (5.7).

$$CMF_{4i} = 1.0 - 0.38 \times P_{ni} \tag{5.7}$$

Where,

CMF_{4i} = crash modification factor for the effect of lighting on total crashes,

P_{ni} = proportion of total crashes for unlighted intersections that occur at night.

The research team assumed the TWSC intersections were lit, as manual verification using Google Street View indicated that a majority of TWSC intersections along the expressways were lit.

The HSM provides a default value of 0.273 for P_{ni} in total crashes if locally derived values are not available.

Assuming the four-legged TWSC intersections have a left-turn lane and a right-turn lane in both approaches of the major road, the associated HSM recommended CMFs are 0.52 and 0.74, respectively.

The CMF values discussed above were applied as multiplicative factors to the total number of crashes derived from Equation (5.5) to obtain the predicted total crashes per year for TWSC intersections.

Table 5.2 presents the five-year crash frequency estimates for various combinations of major- and minor-road AADT levels, as predicted by the negative binomial model developed for TWSC intersections on rural expressways. The predicted five-year crash frequencies were converted to annual crash frequencies by dividing each value by five. Intersection AADT combinations that fall outside the AADT ranges used to develop the negative binomial model are underlined in the table. For these out-of-range AADT conditions, the corresponding five-year crash frequencies are left blank, and HSM-derived values are substituted in the annual frequency column.

Table 5.2 Predicted crash frequency for TWSC intersections

AADT (Major Approach)	AADT (Minor Approach)	Intersection AADT*	5 Years Crash Frequency	Per Year Crash Frequency*
1,500	5,00	2,000		0.12
1,500	2,500	4,000	7.08	1.42
1,500	5,000	6,500	8.69	1.74
1,500	7,500	9,000	10.66	2.13
2,500	5,00	3,000		0.19
2,500	2,500	5,000	7.68	1.54
2,500	5,000	7,500	9.43	1.89
2,500	7,500	10,000	11.57	2.31
5,000	5,00	5,500	8.01	1.60
5,000	2,500	7,500	9.43	1.89
5,000	5,000	10,000	11.57	2.31
5,000	7,500	12,500	14.20	2.84
10,000	5,00	10,500	12.05	2.41
10,000	2,500	12,500	14.20	2.84
10,000	5,000	15,000	17.42	3.48
10,000	7,500	17,500	21.38	4.28
15,000	5,00	15,500	18.15	3.63
15,000	2,500	17,500	21.38	4.28
15,000	5,000	20,000		2.45
15,000	7,500	22,500		2.94
20,000	5,00	20,500		1.12
20,000	2,500	22,500		2.30
20,000	5,000	25,000		3.13
20,000	7,500	27,500		3.75
25,000	5,00	25,500		1.35
25,000	2,500	27,500		2.77
25,000	5,000	30,000		3.78
25,000	7,500	32,500		4.54

Note: *underlined AADT values are outside of the negative binomial model range, and underlined crash frequency is derived from the HSM methodology

It appears that the HSM-derived values tend to underestimate crash frequencies compared to the model developed specifically for Nebraska conditions. On a similar note, Sun et al. (2013) found that the number of crashes at signalized intersections in Missouri was higher than predicted by HSM. Also, in Nebraska, this difference is anticipated, as the HSM base models were derived from aggregated national datasets. In contrast, the Nebraska model reflects the state's unique roadway, traffic, and driver characteristics associated with rural expressways. The relatively higher crash-frequency estimates from the Nebraska-based negative binomial model underscore the importance of applying locally calibrated models when evaluating safety performance on rural expressways. Reliance solely on uncalibrated HSM defaults may underpredict crash occurrence under Nebraska-specific conditions, potentially leading to less conservative design or treatment decisions. While the traffic conditions in Table 5.2 represent generalized intersection scenarios, the HSM model should be calibrated for site-specific applications to account for the individual geometric, traffic, and operational characteristics of each location and to ensure reliable, accurate crash predictions.

Nonetheless, Appendix D includes a table showing estimated crash frequencies at TWSC intersections using a negative binomial model, while disregarding AADT thresholds for readers.

It is important to note that the per-year crash frequency for the TWSC intersection in expressways shown in Table 5.2 will serve as the basis for relative safety performance predictions of alternative designs, such as RCUT and roundabouts, through the application of appropriate CMFs.

5.1.2 Safety Studies of RCUT

The RCUT has emerged as a promising alternative to conventional TWSC intersections along rural expressways. RCUT eliminates direct through and left-turn movements on the minor road; instead, it reroutes minor-road traffic to execute a downstream U-turn via the median on the major road. By eliminating high-conflict crossing maneuvers that commonly cause severe right-angle crashes, RCUTs improve safety performance and operational efficiency on high-speed expressways.

RCUT has gained significant national attention as a proven countermeasure, with more than 250 installations across the United States to date. Several state transportation agencies report substantial reductions in severe crashes following RCUT implementation. Nebraska has recently joined this effort, constructing three RCUT intersections to date, to improve safety along rural expressways located near Murray**,** Humphrey**,** and North Bend. While safety performance observations from RCUT installations in Nebraska to date are encouraging, a more extended period of crash data and additional RCUT sites are required to develop Nebraska-specific SPFs and more reliably quantify their safety effectiveness.

5.1.2.1 Safety Performance Functions for RCUT

While many other studies evaluated the safety performance of RCUTs, a notable study supported by the Florida Department of Transportation (Ulak et al., 2020) developed SPFs for RCUTs using nationwide data. SPFs for all crashes and fatal and injury crashes were developed using 107 unsignalized RCUTs with U-turn lane(s) from 16 states.

Tables 5.3 and 5.4 list the model functions and corresponding coefficients, respectively, for crashes for all severity combined. Tables 5.5 and 5.6 list the model functions and corresponding coefficients for fatal and injury crashes.

Table 5.3 Unsignalized RCUT SPF model functions for all crashes (Ulak et al., 2020)

Model	Function
Model 1	Np = exp (Intercept + $AADT_{major}$ + ln ($AADT_{minor}$) + ln (TOD) + ln (TDLL) + ln (TMeW) + LTLM)
Model 2	Np = exp (Intercept + $AADT_{major}$ + ln ($AADT_{minor}$) + ln (TOD) + ln (TDLL) + LTLM)
Model 3	Np = exp (Intercept + $AADT_{major}$ + ln ($AADT_{minor}$) + ln (TOD) + ln (TDLL))
Model 4	Np = exp (Intercept + $AADT_{major}$ + ln ($AADT_{minor}$) + ln (TOD))
Model 5	Np = exp (Intercept + $AADT_{major}$ + ln ($AADT_{minor}$))

In Table 5.3,

Np = Number of predicted crashes,

$AADT_{major}$ = AADT at major approach,

$AADT_{minor}$ = AADT at minor approach,

TOD = Total offset distance in feet (the total distance between the center of intersection and the U-turn locations for both directions at major road),

TDLL = Total deceleration lane length in feet (the total length of the deceleration lanes before U-turn locations for both directions at major road),

MMeW = Maximum median width in feet (the maximum median width of the major approaches),

LTLM = Number of left-turn lanes from major road.

Table 5.4 Unsignalized RCUT SPF model coefficients (β) for all crashes (Ulak et al., 2020)

Model	Intercept	$AADT_{major}$	$\ln(AADT_{minor})$	ln(TOD)	ln(TDLL)	ln(MMeW)	LTLM
Model 1	-4.884	1.63×10^{-5}	0.433	0.368	-0.091	-0.126	0.623
Model 2	-4.283	1.77×10^{-5}	0.394	0.259	-0.085	-	0.667
Model 3	-3.662	1.78×10^{-5}	0.391	0.263	-0.081	-	-
Model 4	-4.037	2.13×10^{-5}	0.365	0.264	-	-	-
Model 5	-1.852	2.09×10^{-5}	0.350	-	-	-	-

Table 5.5 Unsignalized RCUT SPF model functions for fatal and injury crashes (Ulak et al., 2020)

Model	Function
Model 1	$Np = \exp(\text{Intercept} + \ln(AADT_{major}) + \ln(AADT_{minor}) + \ln(TOD) + \ln(TDLL) + \ln(MMeW))$
Model 2	$Np = \exp(\text{Intercept} + \ln(AADT_{major}) + \ln(AADT_{minor}) + \ln(TOD) + \ln(TDLL))$
Model 3	$Np = \exp(\text{Intercept} + \ln(AADT_{major}) + \ln(AADT_{minor}) + \ln(TDLL))$
Model 4	$Np = \exp(\text{Intercept} + \ln(AADT_{major}) + \ln(AADT_{minor}))$

Table 5.6 Unsignalized RCUT SPF model coefficients (β) for fatal and injury crashes (Ulak et al., 2020)

Model	Intercept	$\ln(AADT_{major})$	$\ln(AADT_{minor})$	ln(TOD)	ln(TDLL)	ln(MMeW)
Model 1	-9.234	0.501	0.265	0.506	-0.133	-0.197
Model 2	-8.648	0.543	0.205	0.343	-0.125	-
Model 3	-5.570	0.515	0.191	-	-0.129	-
Model 4	-6.886	0.599	0.153	-	-	-

Model 5 and Model 4, as presented in Tables 5.3 and 5.5, are simplified models that account only for AADT information in crash predictions. For these two models, CMFs were also developed to account for the geometry of RCUTs. Tables 5.6 and 5.7 list the CMF models and associated coefficients for all crashes and fatal and injury crashes, respectively.

Table 5.7 Unsignalized RCUT CMFs for all crashes (Ulak et al., 2020)

CMF Variable	**Description**	**β**	**SE**	**95% CI of β**	***CMF***	**Reliability**
TDLL	Continuous variable	−0.156	0.088	−0.321 / 0.008	$TDLL^{-0.156}$	High
TOD	Continuous variable	0.158	0.088	−0.007 / 0.323	$TOD^{0.158}$	High
Number of UT	UT = 1:0 UT = 2:1	0.156	0.129	−0.098 / 0.410	1.000 1.169	Moderate
MMeW	Continuous variable	-8.838×10^{-2}	9.341×10^{-2}	-26.77×10^{-2} / 9.257×10^{-2}	$MMeW^{-8.838\times10^{-2}}$	Low
NDW	Along the RCUT	-2.956×10^{-2}	3.863×10^{-2}	-10.48×10^{-2} / 4.534×10^{-2}	$\exp(\beta_{NDW} \times NDW)$	Low
TALL	Continuous variable	5.735×10^{-3}	20.70×10^{-3}	-3.464×10^{-2} / 4.609×10^{-2}	$TALL^{5.735\times10^{-3}}$	Low

Note: UT = U-turns; TALL = total acceleration lane length; NDW = number of driveways, SE= standard error, CI= confidence interval

Table 5.8 Unsignalized RCUT CMFs for fatal and injury crashes (Ulak et al., 2020)

CMF Variable	**Description**	**β**	**SE**	**95% CI of β**	***CMF***	**Reliability**
TOD	Continuous variable	0.305	0.114	0.080 / 0.530	$TOD^{0.305}$	High
TDLL	Continuous variable	−0.263	0.119	−0.490 / −0.034	$TDLL^{-0.263}$	High
MMeW	Continuous variable	−0.163	0.118	−0.395 / 0.078	$MMeW^{-0.163}$	Moderate
NDW	Along the RCUT	-6.799×10^{-2}	5.390×10^{-2}	-0.174/0.037	$\exp(\beta_{NDW} \times NDW)$	Moderate
TALL	Continuous variable	9.632×10^{-3}	29.43×10^{-3}	-4.848×10^{-2} / 6.693×10^{-2}	$TALL^{9.632\times10^{-3}}$	Low
Number of UT	UT = 1:0 UT = 2:1	-4.562×10^{-2}	17.10×10^{-2}	-0.380/0.291	1.000 0.955	Low

Using Model 5 from Table 5.4 (for all crashes) and Model 4 from Table 5.6 (for fatal and injury crashes), the predicted crash frequencies (after the application of CMFs) for various levels of AADT are presented in Table 5.9. The corresponding CMFs from Tables 5.7 and 5.8 were applied as multiplicative adjustment factors to the baseline predicted values (Np) derived from the respective models. These CMFs were calculated using geometric parameter values consistent with the three RCUTs located in Nebraska, including a TOD value of 2,800 feet, a TDDL value of 1,600 feet, an MMeW value of 20 feet, and a UT value of two, ensuring that the CMF adjustments are contextually accurate.

Table 5.9 Predicted crashes at unsignalized RCUTs using Ulak et al. (2020) method

AADT (Major Approach)	AADT (Minor Approach)	Crash Frequency (all crashes)	Crash Frequency (fatal and injury crashes)
1,500	5,00	1.42	0.20
1,500	2,500	2.49	0.26
1,500	5,000	3.17	0.28
1,500	7,500	3.66	0.30
2,500	5,00	1.45	0.27
2,500	2,500	2.54	0.35
2,500	5,000	3.24	0.39
2,500	7,500	3.73	0.41
5,000	5,00	1.53	0.41
5,000	2,500	2.68	0.53
5,000	5,000	3.41	0.59
5,000	7,500	3.94	0.62
10,000	5,00	1.69	0.62
10,000	2,500	2.97	0.80
10,000	5,000	3.79	0.89
10,000	7,500	4.37	0.94
15,000	5,00	1.88	0.80
15,000	2,500	3.30	1.02
15,000	5,000	4.21	1.13
15,000	7,500	4.85	1.20
20,000	5,00	2.09	0.94
20,000	2,500	3.67	1.21
20,000	5,000	4.67	1.34
20,000	7,500	5.38	1.43
25,000	5,00	2.32	1.08
25,000	2,500	4.07	1.38
25,000	5,000	5.19	1.54
25,000	7,500	5.98	1.63

The predicted crash frequencies for RCUTs shown in Table 5.9, derived using the methodology from Ulak et al. (2019), are generally higher than the Nebraska-based TWSC crash frequencies presented in Table 5.2. However, these quantitative values of crash frequency should not be interpreted as a direct, definitive comparison of safety performance between RCUT and TWSC intersections in Nebraska, as the TWSC condition was not used as the baseline when the

SPFs for RCUT were developed. Therefore, the RCUT crash predictions provided here should be regarded only as a general reference point, useful for benchmarking the performance of existing RCUTs or for evaluating proposed RCUT installations. For situations where an RCUT is considered as a retrofit to an existing TWSC intersection, it is more appropriate to apply CMFs developed specifically for TWSC-to-RCUT conversions to estimate the expected change in crash frequency, potentially with greater accuracy.

5.1.2.2 CMFs for Converting TWSC to RCUT

Edara et al. (2013) evaluated five rural unsignalized RCUTs in Missouri and reported a CMF of 0.65 for total crashes, indicating a 35% reduction, and 0.46 for injury crashes, corresponding to a 54% reduction. Notably, no fatal crashes were recorded at the study locations following RCUT implementation. NDOT currently adopts the CMF of 0.652 from this study to estimate crash reductions when converting TWSC intersections to unsignalized RCUTs (NDOT, 2025). Given the geographic proximity of Missouri and Nebraska, as well as the potential similarity in driving behavior and rural highway characteristics, this CMF remains a reasonable and regionally relevant estimate. However, the Missouri study was limited to locations with major-road AADTs ranging from 10,326 to 26,740 and minor-road AADTs from 434 to 1,389, which may limit its applicability to sites with lower major-road volumes or substantially higher minor-road traffic.

More recently, North Carolina, a state recognized for its leadership in RCUT safety research, conducted a statewide evaluation of 31 rural unsignalized RCUTs. The study, published by the NCDOT Traffic Safety Unit in 2023, found a 50% reduction in total crashes and an 80% reduction in frontal-impact crashes, including right-angle collisions (NCDOT Traffic Safety Unit, 2023). Based on these findings, the NCDOT now formally recommends a CMF of

0.50 for converting TWSC intersections to unsignalized Reduced Conflict Intersections (RCIs), a term used interchangeably with RCUTs or their slight design variants.

In 2024, the Missouri DOT published a safety evaluation of 47 rural J-turn (i.e., RCUT) intersections (built between 2005 and 2021) using both a before–after comparison-group analysis and an empirical Bayes analysis (Edara et al., 2024). The comparison-group method included crash data from all 47 RCUT sites, whereas, the empirical Bayes analysis was applied to a subset of 32 sites. The comparison-group analysis found CMFs of 0.56 for total crashes and 0.53 for fatal and injury crashes, while the EB analysis yielded CMFs of 0.60 and 0.49, respectively. Notably, this study encompassed a wider range of traffic volumes, with major-road AADTs from 3,788 to 33,755 and minor-road AADTs from 278 to 12,719. Therefore, CMFs from this study might be more widely applicable than those from the study by Edara et al. (2013), which NDOT currently uses.

However, even though the more recent studies by Edara et al. (2024) suggested a lower CMF value for sites with more diverse AADTs, NDOT's use of the 0.652 value from Edara et al. (2013) is a more conservative approach that helps avoid overestimating expected crash reductions. Using the CMF value of 0.652, Table 5.10 presents estimated annual crash frequencies for intersections converted from TWSC to unsignalized RCUTs.

Table 5.10 Crash frequency estimation of RCUT intersection using CMF

AADT (Major Approach)	AADT (Minor Approach)	Intersection AADT	TWSC Crash Frequency*	Estimated RCUT Crash Frequency using CMF
1,500	5,00	2,000	0.12	0.08
1,500	2,500	4,000	1.42	0.93
1,500	5,000	6,500	1.74	1.13
1,500	7,500	9,000	2.13	1.39
2,500	5,00	3,000	0.19	0.12
2,500	2,500	5,000	1.54	1.00
2,500	5,000	7,500	1.89	1.23
2,500	7,500	10,000	2.31	1.51
5,000	5,00	5,500	1.60	1.04
5,000	2,500	7,500	1.89	1.23
5,000	5,000	10,000	2.31	1.51
5,000	7,500	12,500	2.84	1.85
10,000	5,00	10,500	2.41	1.57
10,000	2,500	12,500	2.84	1.85
10,000	5,000	15,000	3.48	2.27
10,000	7,500	17,500	4.28	2.79
15,000	5,00	15,500	3.63	2.37
15,000	2,500	17,500	4.28	2.79
15,000	5,000	20,000	2.45	1.60
15,000	7,500	22,500	2.94	1.92
20,000	5,00	20,500	1.12	0.73
20,000	2,500	22,500	2.30	1.50
20,000	5,000	25,000	3.13	2.04
20,000	7,500	27,500	3.75	2.45
25,000	5,00	25,500	1.35	0.88
25,000	2,500	27,500	2.77	1.81
25,000	5,000	30,000	3.78	2.46
25,000	7,500	32,500	4.54	2.96

Note: *underlined values represent HSM-based crash frequency for TWSC intersection

It is essential to note that the crash frequency per year listed in Table 5.10 served as the basis for the relative safety performance comparison in terms of economic safety benefits and the corresponding benefit-cost analysis for retrofit decision-making, discussed later in the report.

5.1.3 Safety Studies of Roundabout

Similar to RCUTs, roundabouts are proven safety countermeasures for intersections. Roundabouts are especially effective at reducing severe crashes in high-speed rural areas. While substantial research exists on the safety benefits of converting traditional intersections to single-lane roundabouts, studies focused specifically on converting multilane intersections to roundabouts are relatively limited. Historically, single-lane roundabouts have demonstrated more consistent and greater reductions in crash frequency and severity compared to multi-lane roundabouts. While multi-lane roundabouts reduce the severity of crashes, particularly fatal and injury crashes, their impact on total crash frequency is less pronounced than that of their single-lane counterparts. This difference is often attributed to the complex vehicle interactions introduced by multiple circulating lanes, such as weaving and side-by-side entries, which can contribute to an increased number of PDO crashes.

Table 5.11 presents the crash effects, in terms of CMF, of converting stop-controlled intersections into modern roundabouts (HSM, 2010; Rodegerdts et al., 2007).

Table 5.11 Potential crash effects of converting a stop-controlled intersection into a modern roundabout (HSM, 2010; Rodegerdts et al., 2007)

Treatment	**Setting (Intersection type)**	**Traffic Volume**	**Crash Type (Severity)**	**CMF**	**Standard Error**
Convert intersection with minor-road stop control to modern roundabout	<u>All settings (One or two lanes)</u>	Unspecified	<u>All types (All severities)</u>	<u>0.56</u>	<u>0.05</u>
			All types (Injury)	0.18	0.04
	Rural (One lane)		All types (All severities)	0.29	0.04
			All types (Injury)	0.13	0.04
	Urban (One or two lanes)		All types (All severities)	0.71	0.1
			All types (Injury)	0.19	0.1
	Urban (One lane)		All types (All severities)	0.61	0.1
			All Types (Injury)	0.22	0.1
	Urban (Two lanes)		All types (All severities)	0.88	0.2
	Suburban (One or two lanes)		All types (All severities)	0.68	0.08
			All Types (Injury)	0.29	0.1
	Suburban (One lane)		All Types (All Severities)	0.22	0.07
			All types (Injury)	0.22	0.1
	Suburban (Two lanes)		All types (All severities)	0.81	0.1
			All Types (Injury)	0.32	0.1

Note: NDOT currently adopts the underlined CMF value

Currently, NDOT uses a CMF of 0.56 for converting rural TWSC intersections to multi-lane roundabouts (NDOT, 2025). This CMF represents a 44% reduction in total crash frequency.

Given the limited field data on rural multi-lane roundabout installations, NDOT's use of this CMF provides a conservative but reasonable estimate of expected safety improvements.

Recent evaluations support this CMF. A 2025 Ohio Department of Transportation (Ohio DOT, 2025) study analyzed 76 roundabout sites statewide and found that single-lane roundabouts led to a 69% reduction in injury crashes, while multilane roundabouts achieved an average 25% reduction in total crashes (CMF = 0.75). Despite a smaller reduction in magnitude, multi-lane roundabouts still delivered substantial safety gains at high-risk locations. For example, at a high-speed rural intersection in Clark County, Ohio (State Route 41 and State Route 235), the conversion to a multi-lane roundabout resulted in zero reported fatal crashes in the 11 years following its installation, despite a prior history of severe crash occurrences.

The North Carolina DOT, in its CMF guidance, lists a CMF of 0.16 for all crashes and a CMF of 1.71 for property-damage-only (PDO) crashes when converting a TWSC intersection to a multi-lane roundabout (North Carolina DOT, 2025). These numbers suggest an 84% reduction in injury crashes but a 71% increase in PDO crashes. However, North Carolina DOT labels these CMFs as "interim," indicating the need for further study to establish more reliable values. This aligns with several studies that have observed increases in PDO crashes following multilane roundabout conversions.

Khattak et al. (2014), who cited findings from a Wisconsin study (Bill et al., 2011) showing that multi-lane roundabouts led to a 41.3% (CMF=0.58) reduction in total crashes and a 67.9% (CMF=0.32) reduction in injury crashes. These findings reinforce the general consensus that multilane roundabouts can deliver significant safety benefits, though often with smaller overall crash reductions than single-lane designs.

Considering the above discussions, NDOT's use of a CMF of 0.56 for multilane roundabout conversions appears to strike a balance between over- and underestimating crash impacts for multilane roundabouts converted from TWSC. Table 5.12 provides estimated annual crash frequencies for multi-lane roundabouts using a CMF of 0.56, compared with baseline TWSC crash frequencies for Nebraska rural intersections at expressways.

Table 5.12 Crash frequency estimation of multi-lane roundabouts using CMF

AADT (Major Approach)	**AADT (Minor Approach)**	**Intersection AADT**	**TWSC Crash Frequency***	**Estimated Multi-Lane Roundabout Crash Frequency using CMF**
1,500	5,00	<u>2,000</u>	<u>0.12</u>	0.07
1,500	2,500	4,000	1.42	0.80
1,500	5,000	6,500	1.74	0.97
1,500	7,500	9,000	2.13	1.19
2,500	5,00	<u>3,000</u>	<u>0.19</u>	0.11
2,500	2,500	5,000	1.54	0.86
2,500	5,000	7,500	1.89	1.06
2,500	7,500	10,000	2.31	1.29
5,000	5,00	5,500	1.60	0.90
5,000	2,500	7,500	1.89	1.06
5,000	5,000	10,000	2.31	1.29
5,000	7,500	12,500	2.84	1.59
10,000	5,00	10,500	2.41	1.35
10,000	2,500	12,500	2.84	1.59
10,000	5,000	15,000	3.48	1.95
10,000	7,500	17,500	4.28	2.40
15,000	5,00	15,500	3.63	2.03
15,000	2,500	17,500	4.28	2.40
15,000	5,000	<u>20,000</u>	<u>2.45</u>	1.37
15,000	7,500	<u>22,500</u>	<u>2.94</u>	1.65
20,000	5,00	<u>20,500</u>	<u>1.12</u>	0.63
20,000	2,500	<u>22,500</u>	<u>2.30</u>	1.29
20,000	5,000	<u>25,000</u>	<u>3.13</u>	1.75
20,000	7,500	<u>27,500</u>	<u>3.75</u>	2.10
25,000	5,00	<u>25,500</u>	<u>1.35</u>	0.76
25,000	2,500	<u>27,500</u>	<u>2.77</u>	1.55
25,000	5,000	<u>30,000</u>	<u>3.78</u>	2.12
25,000	7,500	<u>32,500</u>	<u>4.54</u>	2.54

*underlined values represent HSM-based crash frequency for the TWSC intersection

The crash frequency per year listed in Table 5.12 will serve as the basis for the relative safety performance comparison in terms of economic safety benefits and the corresponding benefit-cost analysis for decision-making regarding converting a TWSC intersection to a multi-lane roundabout.

5.2 Safety Studies of Interchanges

The FHWA study, "Safety Comparisons Between Interchange Types" (Report Number: FHWA-HRT-23-049), developed a planning-level safety assessment tool and an interchange safety comparison process for FHWA and State DOTs (Himes et al., 2023a; Zhang and Himes, 2025). The safety assessment tool will allow agencies to quantify the safety performance of proposed designs against a base or reference condition. This study focused on the most frequently considered service interchange configurations, comprising more than 75% of Intersection Justification Reports (IJRs). It developed a planning-level predictive model and a spreadsheet implementation tool that allows practitioners to estimate crash frequency and severity for alternative interchange layouts. Interchange types included in the study were Conventional Diamond Interchange (DI), Compressed Diamond Interchange (CDI), Tight Diamond Interchange (TDI), DDI, Single Point Diamond Interchange (SPDI), Parclo A, Parclo B, and Parclo (AB) as shown in Figure 5.5.

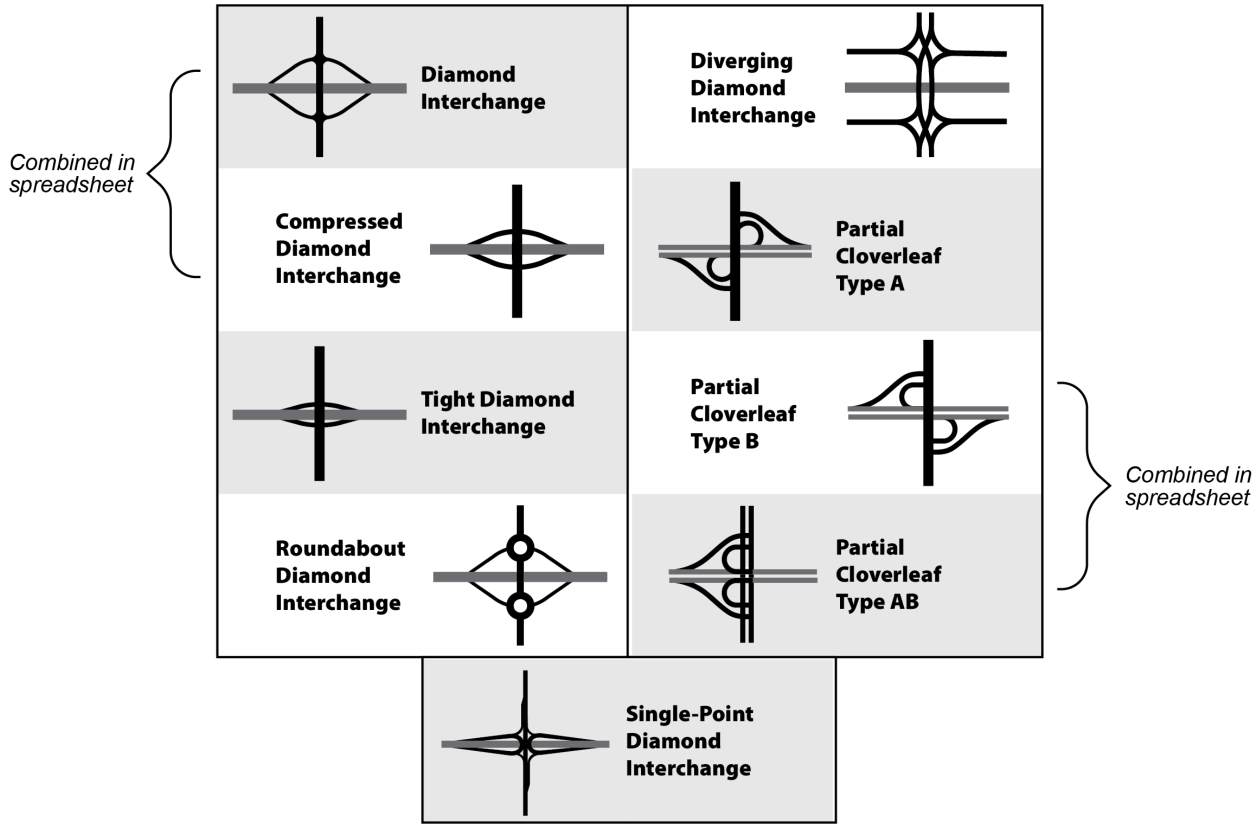


Figure 5.5 Interchange configurations included in the FHWA spreadsheet tool (Himes et al. 2023b)

A DI includes relatively straight entrance and exit ramps leading from the mainline to the crossroad. Two ramp terminals are spaced at least 800 ft apart. On the other hand, for CDI, the ramp terminals are spaced between 400 and 800 feet. For TDI, the terminals are spaced 200-400 feet apart (Himes et al., 2023a).

Inputs to the spreadsheet model include freeway and cross-street AADT, ramp AADT, number of lanes, area type (urban/rural), skew of the interchange, presence of managed lanes, number of left-turn lanes on the cross-road at the terminals, variation in ramp volumes, and adjacency to other interchanges or gore areas. The model provides predictions of crash frequencies for different levels of severity, such as PDO (property damage only) and KABC (K-fatal injury, A-suspected serious injury, B-suspected minor injury, C-possible injury), and distributions at KA, B, and C levels (Himes et al., 2023b).

The spreadsheet tool focuses on the interchange area itself rather than the broader interchange influence area. To maintain consistency and eliminate variability across all nine interchange configurations analyzed, the interchange area applicable to the predictive model developed was defined as follows (Himes et al., 2023b):

- Mainline: Includes 1,500 ft upstream and downstream of the painted gores farthest from the crossroad on either side.
- Cross-street: Includes 100 ft upstream and downstream of the gore or curb return of the outermost ramp connection for each terminal (cross-street).

Figure 5.6 shows the interchange study area.

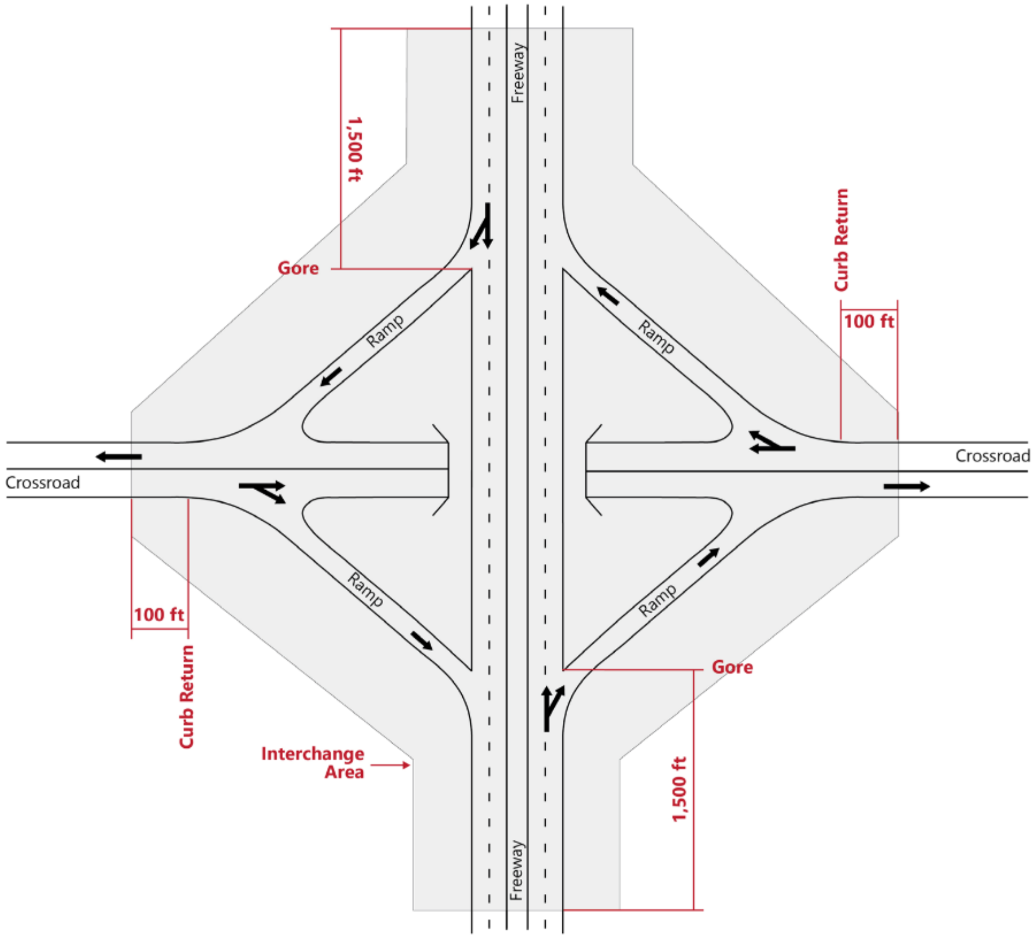


Figure 5.6 Definition of interchange area for the predictive models (Himes et al., 2023b)

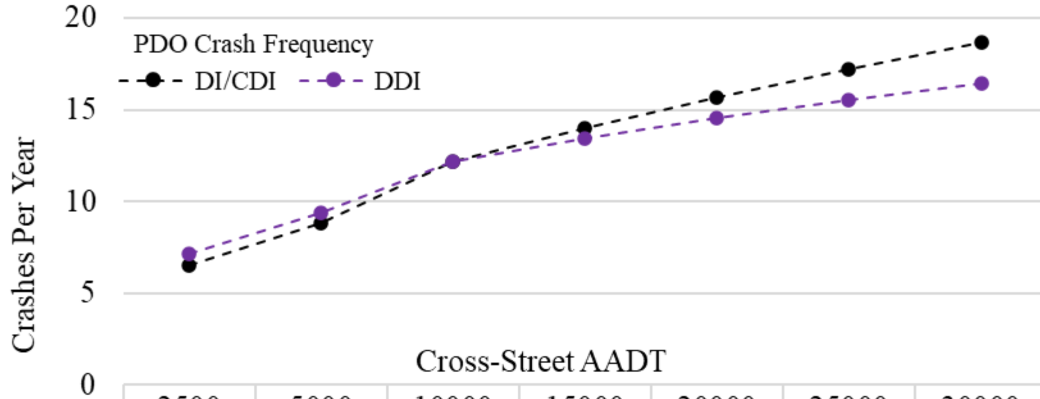
The crash prediction models were developed using DI/CDI as the base condition, and other interchange types, including DDI crashes, were predicted accordingly. Table 5.13 shows the applicability ranges of traffic volumes for mainline/freeway/expressways, cross-street, and ramps locations for different interchange types.

Table 5.13 Traffic volume ranges applicable for predictive models (Himes et al., 2023b)

Interchange Component	DI	CDI	Roundabout DI	DDI	Parclo Type AB	Parclo Type A or B	SPDI	TDI
Mainline AADT (minimum)	5,000	23,100	5,000	29,000	5,500	6,400	21,000	17,000
Mainline AADT (maximum)	210,000	236,000	94,600	191,000	300,000	144,000	261,000	207,300
Cross-street AADT (minimum)	350	11,000	750	2,000	200	200	3,700	3,200
Cross-street AADT (maximum)	40,500	52,900	22,500	47,000	57,000	68,000	64,000	55,000
Total entrance ramp volume (minimum)	100	6,800	850	2,000	200	50	3,100	4,000
Total entrance ramp volume (maximum)	33,400	25,500	20,400	38,500	29,100	34,400	70,000	36,500
Total exit ramp volume (minimum)	125	42,50	800	2,000	175	50	3,200	4,500
Total exit ramp volume (maximum)	24,500	22,500	20,900	45,000	27,200	39,600	75,000	36,700

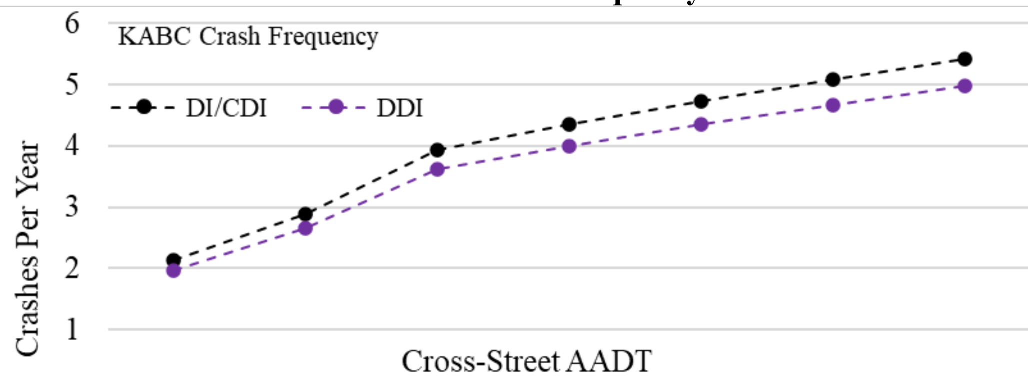
Similar to the traffic scenarios analyzed for the interchange discussed in Chapter 4, the crash implications of converting DI/CDI to DDI were studied using this tool (Himes et al., 2023b). Therefore, by complying with the application threshold listed in Table 5.13, the DDI safety performance, in terms of crash frequency per year, were predicted for three levels of ramp AADT (i.e., 2,500; 5,000; 7,500) and seven levels of cross-street AADT (i.e., 2,500; 5,000; 10,000; 15,000; 20,000; 25,000; 30,000). The mainline AADT was assumed to be 30,000.

Furthermore, it was assumed that the turning ratios for cross-street AADT were distributed as 40% left-turn movements, 40% through movements, and 20% right-turn movements.



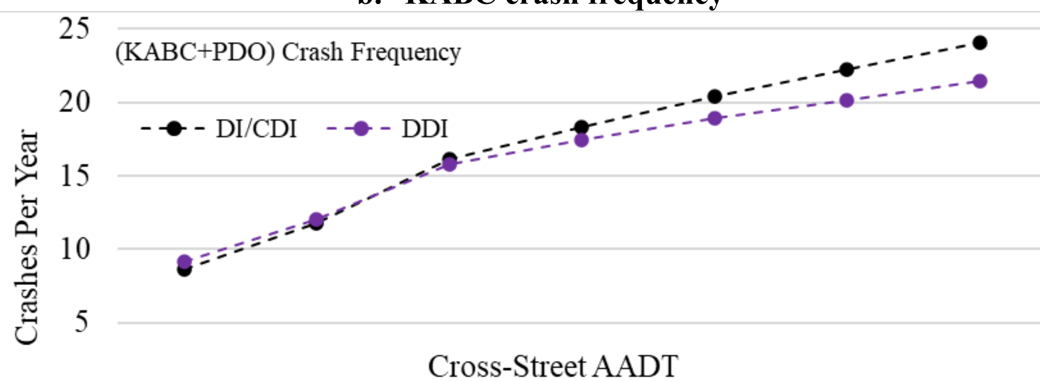
	2500	5000	10000	15000	20000	25000	30000
DI/CDI	6.53	8.85	12.17	14.01	15.67	17.21	18.68
DDI	7.19	9.38	12.20	13.45	14.54	15.52	16.43

a. PDO crash frequency



	2500	5000	10000	15000	20000	25000	30000
DI/CDI	2.13	2.90	3.93	4.34	4.72	5.08	5.42
DDI	1.96	2.67	3.62	4.00	4.35	4.67	4.99

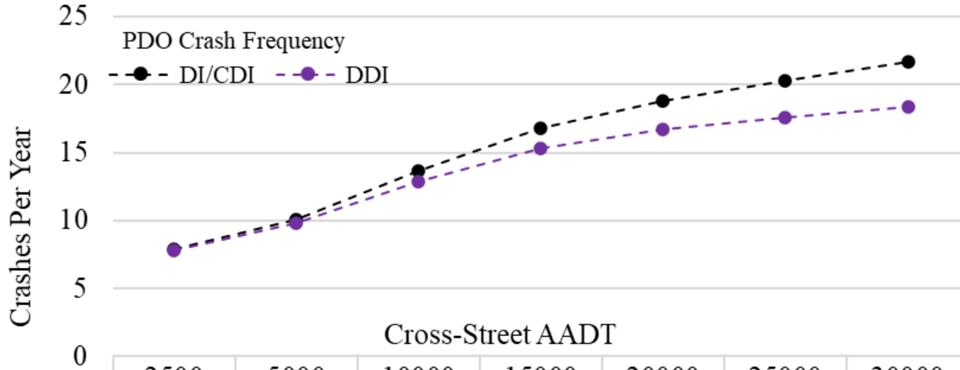
b. KABC crash frequency



	2500	5000	10000	15000	20000	25000	30000
DI/CDI	8.66	11.75	16.10	18.35	20.40	22.29	24.10
DDI	9.14	12.05	15.82	17.45	18.89	20.19	21.42

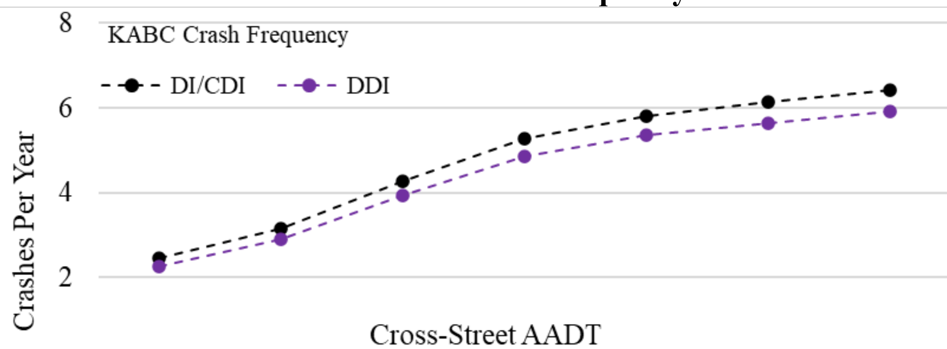
c. (KABC+PDO) crash frequency

Figure 5.7 Predicted crash frequencies for DI/CDI and DDI for 2,500 ramp AADT



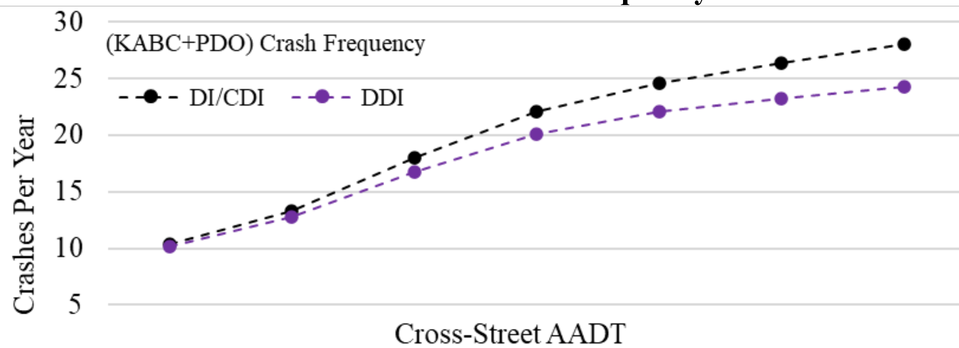
	2500	5000	10000	15000	20000	25000	30000
DI/CDI	7.89	10.10	13.69	16.81	18.83	20.29	21.67
DDI	7.84	9.83	12.84	15.29	16.69	17.58	18.40

a. PDO crash frequency



	2500	5000	10000	15000	20000	25000	30000
DI/CDI	2.46	3.15	4.28	5.28	5.81	6.13	6.43
DDI	2.26	2.90	3.94	4.86	5.35	5.64	5.92

b. KABC crash frequency



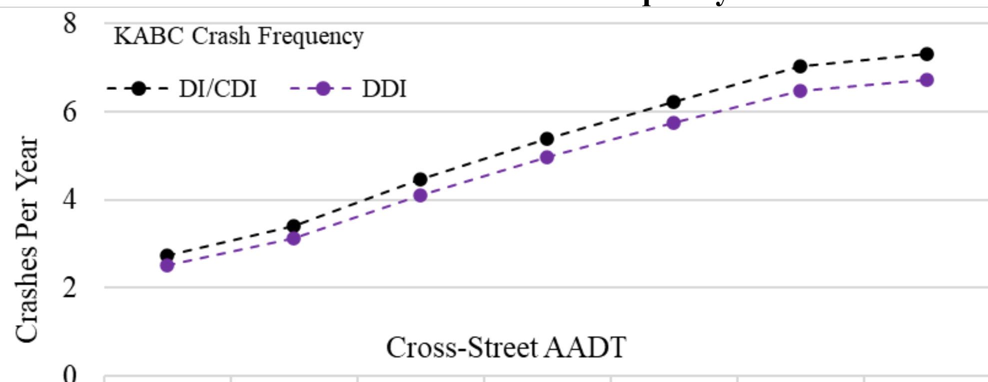
	2500	5000	10000	15000	20000	25000	30000
DI/CDI	10.35	13.25	17.97	22.09	24.64	26.42	28.09
DDI	10.11	12.73	16.78	20.15	22.04	23.22	24.32

c. (KABC+PDO) crash frequency

Figure 5.8 Predicted crash frequencies for DI/CDI and DDI for 5,000 ramp AADT

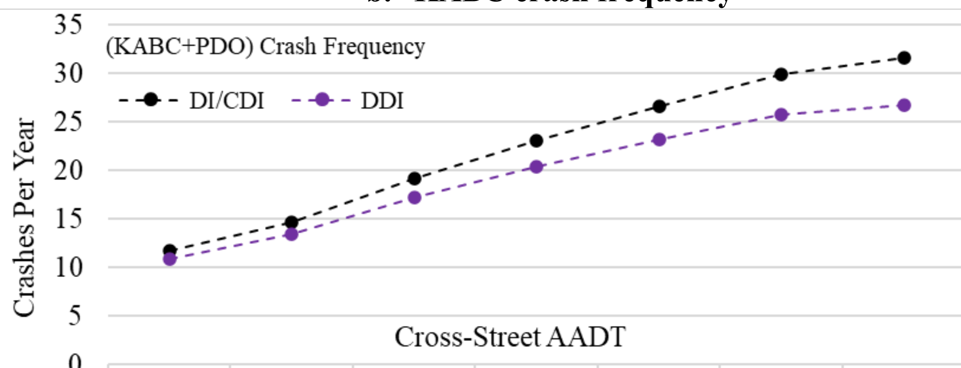
	2500	5000	10000	15000	20000	25000	30000
DI/CDI	8.99	11.21	14.68	17.67	20.40	22.95	24.30
DDI	8.39	10.30	13.12	15.42	17.44	19.26	20.05

a. PDO crash frequency



	2500	5000	10000	15000	20000	25000	30000
DI/CDI	2.74	3.40	4.46	5.39	6.24	7.03	7.31
DDI	2.52	3.13	4.11	4.96	5.74	6.47	6.73

b. KABC crash frequency



	2500	5000	10000	15000	20000	25000	30000
DI/CDI	11.73	14.62	19.14	23.05	26.63	29.98	31.61
DDI	10.91	13.43	17.23	20.38	23.18	25.73	26.78

c. (KABC+PDO) crash frequency

Figure 5.9 Predicted crash frequencies for DI/CDI and DDI for 7,500 ramp AADT

Figures 5.7, 5.8, and 5.9 illustrate the predicted annual crash frequencies for DI/CDI and DDI configurations at ramp AADTs of 2,500, 5,000, and 7,500, respectively. Each figure presented results for PDO crashes**,** KABC crashes**,** and all crashes combined (i.e., PDO and KABC crashes). While these figures depicted the overall trend of increasing crash frequency as cross-street AADT rose from 1,500 to 30,000, they also contained embedded data tables listing the corresponding numerical values of crash frequency.

A clear trend across all scenarios is that crash frequency increased with both mainline and ramp AADT, as expected. Furthermore, the difference in predicted crash frequencies between DI/CDI and DDI configurations became more pronounced as the cross-street and ramp AADTs increased. In other words, the relative safety advantage of the DDI over the DI/CDI tended to increase as traffic volumes on both the cross-street and ramp approaches increased.

Table 5.14 further disaggregates crash frequency by KA, B, and C injury types for DI/CDI and DDI configurations across all AADT combinations for cross-street and ramp.

Table 5.14 Predicted crash frequency (per year) distribution of KA, B, and C injury

AADT		KA Crash Frequency		B Crash Frequency		C Crash Frequency	
Ramp	Cross-street	DI/CDI	DDI	DI/CDI	DDI	DI/CDI	DDI
2,500	2,500	0.21	0.11	0.65	0.43	1.27	1.42
	5,000	0.28	0.15	0.88	0.59	1.74	1.93
	10,000	0.38	0.20	1.20	0.79	2.36	2.62
	15,000	0.42	0.22	1.32	0.88	2.60	2.90
	20,000	0.46	0.24	1.44	0.95	2.83	3.15
	25,000	0.49	0.26	1.55	1.03	3.04	3.39
	30,000	0.52	0.28	1.65	1.09	3.25	3.62
5,000	2,500	0.24	0.13	0.75	0.50	1.47	1.64
	5,000	0.30	0.16	0.96	0.64	1.89	2.10
	10,000	0.41	0.22	1.30	0.87	2.57	2.86
	15,000	0.51	0.27	1.61	1.07	3.16	3.53
	20,000	0.56	0.30	1.77	1.17	3.48	3.88
	25,000	0.59	0.31	1.87	1.24	3.67	4.09
	30,000	0.62	0.33	1.96	1.30	3.85	4.29
7,500	2,500	0.26	0.14	0.83	0.55	1.64	1.83
	5,000	0.33	0.17	1.04	0.69	2.04	2.27
	10,000	0.43	0.23	1.36	0.90	2.67	2.98
	15,000	0.52	0.27	1.64	1.09	3.23	3.60
	20,000	0.60	0.32	1.90	1.26	3.74	4.16
	25,000	0.68	0.36	2.14	1.42	4.21	4.69
	30,000	0.71	0.37	2.22	1.48	4.38	4.88

The model incorporated in the spreadsheet tool does not provide the distribution of crash frequencies between K and A severity levels. Since one of the primary objectives of this safety analysis was to perform an economic evaluation of crash impacts, and because the unit costs of K and A crashes differed significantly, it was necessary to estimate an appropriate proportion between these two severity categories.

First, the combined KA crash savings for DDI relative to DI/CDI were computed. To disaggregate these combined KA crash savings into individual K and A components, the research team adopted crash proportion findings from Abdelrahman et al. (2021), who analyzed 80 DDI sites and 240 conventional DI sites to develop CMFs for DI-to-DDI conversions. Based on the 80 DDI sites, the observed crash frequency proportion of K: KABC was 0.00783:1. Using this ratio, the predicted K crash frequency was estimated by applying the 0.00783 factor to the total KABC crashes for DDI, and the A crash frequency was calculated as the remaining portion of the KA crash savings after subtracting the K crashes.

Table 5.15 Crash savings from DDI configuration compared to DI/CDI

AADT		**Crash Savings Per Year by DDI***						
Ramp	**Cross-street**	**Total**	**KA**	**K**	**A**	**B**	**C**	**PDO**
2,500	2,500	-0.49	0.10	0.02	0.08	0.22	-0.15	-0.65
	5,000	-0.31	0.13	0.02	0.11	0.30	-0.20	-0.54
	10,000	0.28	0.18	0.03	0.15	0.40	-0.27	-0.03
	15,000	0.90	0.20	0.03	0.17	0.44	-0.30	0.56
	20,000	1.51	0.21	0.03	0.18	0.48	-0.32	1.13
	25,000	2.10	0.23	0.04	0.19	0.52	-0.35	1.70
	30,000	2.68	0.25	0.04	0.21	0.55	-0.37	2.25
5,000	2,500	0.24	0.11	0.02	0.09	0.25	-0.17	0.04
	5,000	0.52	0.14	0.02	0.12	0.32	-0.22	0.27
	10,000	1.19	0.19	0.03	0.16	0.44	-0.29	0.85
	15,000	1.94	0.24	0.04	0.20	0.54	-0.36	1.52
	20,000	2.60	0.26	0.04	0.22	0.60	-0.40	2.14
	25,000	3.19	0.28	0.04	0.23	0.63	-0.42	2.71
	30,000	3.78	0.29	0.05	0.25	0.66	-0.44	3.26
7,500	2,500	0.82	0.12	0.02	0.10	0.28	-0.19	0.60
	5,000	1.18	0.15	0.02	0.13	0.35	-0.23	0.91
	10,000	1.91	0.20	0.03	0.17	0.46	-0.31	1.56
	15,000	2.67	0.25	0.04	0.21	0.55	-0.37	2.25
	20,000	3.46	0.28	0.04	0.24	0.64	-0.43	2.96
	25,000	4.25	0.32	0.05	0.27	0.72	-0.48	3.70
	30,000	4.83	0.33	0.05	0.28	0.75	-0.50	4.25

*Positive values indicate crash reduction by DDI

Table 5.15 presents the estimated annual crash savings associated with the DDI configuration compared to the DI/CDI configuration for total crashes and by crash severity level, including KA, K, A, B, C, and PDO crashes**.** In two instances, the total crash frequency was found to increase under the DDI configuration. This increase was attributed to a rise in PDO or other lower-severity crashes. However, the more severe crash types, specifically K and A injuries, showed notable reductions. These reductions are particularly important because the economic impact of K and A crashes is substantially higher than that of PDO or non-injury crashes. As a result, even with a slight increase in lower-severity crashes, the DDI configurations are expected to yield overall economic benefits**.** A detailed analysis of these comparisons was presented in subsequent chapters, where benefit-cost evaluations and design guideline recommendations were discussed.

The total (i.e., PDO plus KABC) crash reduction by DDI across 21 scenarios shown in Table 5.15 ranged from -10% to 18%, corresponding to equivalent CMFs of 1.10 to 0.82. The average CMF from 21 scenarios is 0.92. Currently, NDOT uses a CMF value of 0.633 to convert signalized DI to DDI for “all” crash types (NDOT, 2025). This value was adopted from the study by Nye et al. (2019), which conducted a comparison-group analysis using 26 DDIs from 11 different states. Another recent study by Abdelrahman et al. (2021), discussed earlier, used 80 DDIs to develop both SPFs and CMFs. This study found CMFs of 0.736 from the comparison-group analysis and 0.858 from the empirical Bayes (full-sample) analysis. Therefore, it appears that the FHWA “Interchange Comparison” spreadsheet tools used in this project yield the most conservative CMFs compared to those in the other two studies.

The research team used the outcomes of the FHWA “Interchange Comparison Safety Tool” for the safety analysis, the corresponding economic evaluation, and the guidelines for this project. This tool appears to yield the most conservative CMFs compared with previously published studies. This conservative approach enhances confidence that the crash-reduction benefits estimated here are not overestimated. Furthermore, the FHWA tool's safety outcomes are sensitive to variations in AADT. Therefore, the research team adopted the FHWA tool outputs as the basis for the safety analysis, economic evaluation, and guideline development presented in this report.

Chapter 6 Operational and Safety Studies of Signalized Intersections

While unsignalized intersections along rural expressways generally have adequate capacity to accommodate existing traffic volumes, signalization may be warranted at certain rural or suburban locations where traffic demand is relatively higher and operational efficiency becomes a concern. This chapter examined the operational performance of signalized intersections in comparison to their unsignalized counterparts and evaluates the potential safety impacts associated with installing signal control.

6.1 Operational Studies of Signalized Intersection

The research team collected traffic operation data from a four-legged signalized intersection (at the junction of Hwy 77 and W 1st St) at Wahoo, Nebraska, during September 2024, as shown in Figure 6.1.

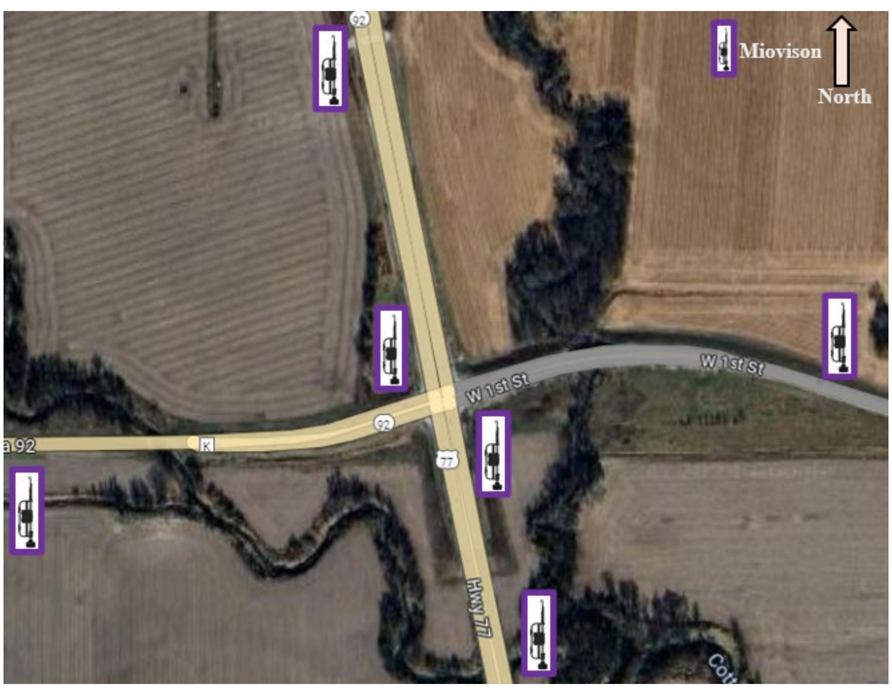


Figure 6.1 Data collection at four-legged signalized intersections at Wahoo, Nebraska

The intersection had AADT of 8,000 and 4,500 for the major and minor approaches, respectively (NDOT AADT, 2025). The speed limits were 60 mph and 50 mph, respectively, for the four-lane and two-lane highways forming the intersection. The signal timing data were collected from NDOT traffic divisions and also captured through video recordings.

A similar approach to data collection using video cameras, LiDAR speed guns, and MAC addresses was maintained for the Wahoo intersection, as discussed for the unsignalized intersections in Chapter 3. Using the calibration methodology presented in Figure 3.3 (Chapter 3), parameters from the Wiedmann 74 model and the “priority rules” functionality in VISSIM were used for model calibration. The final calibrated simulation parameters for signalized intersections are presented in Table 6.1.

Table 6.1 Final calibrated model parameters for signalized intersection

Calibrated Simulation Parameters Values			
	Parameters	**PC**	**T**
Car-following behaviors	AX	7.5	9.0
	BX_add.	2.25	3.25
	BX_mult.	4.25	4.75
Minimum gap acceptance (s)	Permissive Left-turn from major	5.3	7.2
	Right-turn from minor	6.3	7.6

Using the calibrated simulation parameters, sensitivity analyses were performed with various traffic scenarios and the same lane configurations described in Section 3.1.5, consistent with the unsignalized intersection analysis. The objective was to compare the operational performance of signalized and unsignalized intersections under identical lane configurations to ensure a valid comparison. At rural intersections, the minor approach typically lacks dedicated left-turn bays, as observed at all TWSC sites. However, the operational performance of signalized intersections is highly influenced by lane configuration and signal phasing. Additionally, for intersections with relatively low traffic volumes, signalization may not be appropriate, as peak-hour operations require multiple signal phases that interrupt major-road traffic to accommodate minor movements. Moreover, implementing signal control along high-speed rural corridors can increase crash risk, as discussed in the following section.

From the sensitivity analysis of signalized intersections, a minor-road AADT of 7,500 (or above 5,000) resulted in substantial delays on the minor approach, often leading to an overall intersection LOS of E or F during peak hours. In contrast, TWSC intersections accommodated minor-road AADTs between 5,000 and 7,500 while maintaining overall intersection LOS between B and D for major-road AADTs up to 5,000. Roundabouts performed even better, handling minor-road AADTs of 7,500 for major-road AADTs up to 15,000 with intersection LOS between A and C, except under limited conditions involving 60 percent left-turn volumes at 15,000 AADT. Overall, both TWSC and roundabout configurations outperformed signalized intersections when the major-road AADT exceeded 5,000, for the minor-road AADT discussed. Therefore, for signalized intersections, it is recommended that left-turn bays be provided on the minor approach legs when minor-road AADT exceeds 5,000 to improve operational performance.

When the minor-road AADT was below 5,000, the TWSC intersection maintained an overall intersection LOS of A to B for major-road AADT values up to 10,000 during peak-hour periods. The roundabout, on the other hand, performed better by sustaining LOS A to B for major-road AADT values up to 15,000, except for a few scenarios involving 60% left-turn movements at 15,000 major-road AADT. In comparison, the signalized intersection maintained LOS ranging from B to D for major-road AADT values up to 20,000 when the minor-road AADT was less than 5,000. These findings suggest that, from an operational standpoint, the signalized intersection may begin to outperform the TWSC intersection once the major-road AADT exceeds approximately 10,000 and the minor-road AADT remains moderate without a left-turn bay on the minor approach. Compared to the roundabout, signalization may become more advantageous when the major-road AADT reaches or exceeds 15,000. For major-road AADT values around 25,000, a left-turn bay on the minor approach is necessary for the signalized intersection, even under moderate minor-road AADT conditions.

It is important to note that roundabouts generally perform better when AADT values on the major and minor approaches are relatively balanced. For example, under balanced traffic conditions such as AADT of 2,500 or 5,000 on each approach, the roundabout-maintained intersection LOS is A. In contrast, signalized intersections operating under similar AADT conditions provided LOS C to D. This is one of the key reasons that conversion from a signalized intersection to a roundabout is relatively common in practice, as it offers both operational improvement and enhanced safety performance. However, when AADT becomes highly unbalanced, such as when the major road volume greatly exceeds that of the minor approach, roundabout performance tends to deteriorate due to increased entry delays on the minor approaches. In such cases, signalized intersections may provide better overall LOS consistency under highly directional flow conditions

Roadway segments of expressways in rural Nebraska typically do not experience traffic volumes exceeding 15,000 AADT, making unsignalized intersections a more practical option than signalized intersections. Therefore, signalization should be considered only at specific sites where local traffic conditions warrant it. In general, comparisons among unsignalized intersection types remain the primary focus for intersection selection decisions along rural expressways in Nebraska.

6.2 Safety Studies of Signalized Intersection

The installation of traffic signals at unsignalized intersections is known to reduce angled crashes, but it can also increase rear-end crashes. In terms of overall, CMF for all crashes (all severities and all types), the safety implications are not well understood.

The HSM (2010) listed CMFs for converting minor-road stop-controlled intersections to signalized intersections as shown in Table 6.2.

Table 6.2 Potential crash effects of converting from minor road stop-controlled intersection to signal-controlled intersections (HSM, 2010; Davis and Aul, 2007; Harkey et al., 2008)

Treatment	**Setting (Intersection Type)**	**Traffic Volume AADT (veh/day)**	**Crash Type (Severity)**	**CMF (Std. Error)**
Install a traffic signal	Urban (major road speed limit at least 40 mph; four leg)	Unspecified	All types (All severities)	0.95* (0.09)
			Right-angle (All severities)	0.33 (0.06)
			Rear-end (All severities)	2.43 (0.40)
	Rural (three leg and four leg)	Major road 3,261 to 29,926; Minor road 101 to 10,300	All types (All severities)	0.56 (0.03)
			Right-angle (All severities)	0.23 (0.02)
			Left-turn (All severities)	0.40 (0.06)
			Rear-end (All severities)	1.58 (0.20)

*crash may decrease, increase, or remain the same.

Table 6.2 shows that for urban areas, signalization may increase, decrease, or will not change the total crashes when converted from minor road stop-controlled intersections, as the CMF of 0.95 with a standard error of 0.9 suggests CMF may range from 0.85 to 1.13 (the confidence interval defined by the CMF ± two times the standard error).

In Table 6.2, HSM (2010) listed CMF for rural intersections based on the NCHRP Report 617 (Harkey et al., 2008). Harkey et al. (2008) investigated the safety effects of converting rural stop-controlled intersections to signalized intersections. Using an empirical Bayes before-and-after methodology, the researchers analyzed 45 converted intersections in California (1993-2002) and Minnesota (1991-2002), with a comparison group of approximately 3,500 intersections. The study found that installation of traffic signals reduced total crashes by 44 percent (CMF = 0.56), right-angle crashes by 77 percent (CMF = 0.23), and left-turn crashes by 60 percent (CMF = 0.40), while rear-end crashes increased by 58 percent (CMF = 1.58). The authors noted little difference between the effects at three-leg vs. four-leg sites or between sites with two lanes on the major vs. four lanes. Thus, the CMF may be applied to all rural site types. One limitation of the Harkey et al. (2008) study is that all evaluated intersections dated from 1993 to 2002, which may not fully reflect current roadway and traffic conditions. A more recent investigation would likely provide results more representative of today's rural expressway environments. Harkey et al. (2008) also reported that the Iowa DOT provided a dataset of high-speed rural intersections that were converted from stop control to signal control, along with a reference group of comparable sites. Although the Iowa dataset was too limited to establish the time trends necessary for a full empirical Bayes analysis, a preliminary review showed similar directional effects to those found in California and Minnesota, particularly for right-angle, left-turn, and rear-end crashes. However, the Iowa analysis produced a CMF of 0.95 for total crashes with a standard error of 0.085, indicating the actual CMF could range from 0.78 to 1.12. This suggests that the safety effect of signal installation in Iowa may not be statistically conclusive. Nonetheless, the Iowa findings are particularly relevant to Nebraska because of the geographic proximity and the comparable characteristics of rural expressways in both states.

Recently, the North Carolina DOT used a CMF of 0.81 to convert a TWSC to a signalized intersection (Hummer, 2024). In developing the Safest Feasible Intersection Design (SaFID), Hummer (2024) applied a technique known as CMF chaining. The approach assumes that if a CMF is available for converting condition a to condition b, and another CMF exists for converting b to c, and if these CMFs are independent of each other, then multiplying the CMF for a to b by the CMF for b to c yields an approximate CMF for a to c without substantial loss of accuracy. NDOT applies a CMF of 0.56 for converting a TWSC intersection to a multi-lane roundabout and a CMF of 0.522 for converting a signalized intersection to a multi-lane roundabout for rural areas (NDOT CMF, 2025). Based on the chaining relationship, the implied CMF for converting a TWSC intersection to a signalized intersection is approximately 1.08. This means that conversion from TWSC to a signalized intersection may increase crashes in Nebraska.

Wang and Abdel-Aty (2014) conducted a safety evaluation of installing signals at stop-controlled intersections for different AADT levels in Florida. The authors recommended considering these CMFs beside the signal warrants discussed in MUTCD (2023). The CMF values from this study are included in the CMF Clearinghouse (2025). Wang and Abdel-Aty (2014) found CMFs of 0.656, 0.502, 1.119, 0.76, and 0.768 for major-road AADT (area type not specified) from 0 to 10,000, 10,000 to 20,000; 20,000 to 25,000; 25,000 to 35,000; and beyond 35,000; respectively. This study used the before-after method with the empirical Bayes method for data spanning 2004-2009. Notably, for 20,000 to 25,000 major-road AADT, the CMF value of 1.119 indicated an increase in crashes with signal installations.

Returning to the Iowa-based study, the Iowa DOT published planning-level CMF values for different treatments. It listed a CMF value of 1.0 when a countermeasure of installing a traffic signal is applied to an existing unsignalized intersection (Iowa DOT CMF, 2025). Therefore, Iowa DOT assumes no positive or negative crash impacts for installing signalization. This approach supported the findings of Iowa DOT previously discussed in Harkey et al. (2008). Most recently, the Iowa DOT conducted a comprehensive study on the safety impacts of installing traffic signals at stop-controlled intersections, focusing on high-speed conditions specific to Iowa (Abukhalil et al., 2025). The study aimed to address limitations of earlier national research, which produced mixed findings.

Abukhalil et al. (2025) defined a high-speed intersection as an intersection where at least one leg had a speed limit of 45 mph or higher. The study used 73 treatment sites (i.e., signalized intersections) and 813 reference sites (i.e., unsignalized intersections). The study found that all crashes (i.e., KABCO) increased to a CMF value of 1.795 for all facility types. The CMFs for 3-legged intersections, 4-legged intersections, divided median intersections, and undivided intersections were 1.504, 1.730, 2.023, and 0.798, respectively. The study further attempted different validations of CMFs. The final recommendations regarding signalization were as follows (Abukhalil et al., 2025; CITRE 2025):

- Installation of traffic signals at high-speed intersections is generally predicted to result in an overall crash increase ranging from 45% to 100%, with mainline rear-end crashes rising by 70% to 340%. These rear-end crashes are also more severe due to higher approach speeds.

- Broadside or right-angle crashes typically decrease by about 50% to 60% after signalization, though some may persist due to red-light violations or permissive left-turn movements.
- Notable overall crash reductions were observed only at undivided intersections, likely because these sites are more common in suburban or urban transition areas where drivers anticipate signal control.
- Prior to recommending signal installation at high-speed intersections, alternative treatments such as roundabouts, reduced-conflict intersections or right-in/right-out designs (e.g., RCUT) should be carefully evaluated.

To further evaluate the safety implications of signalization, the research team estimated crash frequencies using the SPFs provided in the HSM (2010) for a base condition of a four-legged TWSC intersection (discussed in Section 5.1.1.2 in this report) and for a four-legged signalized intersection (no base condition provided in HSM). The ratio of crash frequency between the TWSC and signalized intersections was calculated for 28 different traffic volume combinations. The average ratio was found to be 2.33, as detailed in Appendix E. This indicates that, on average, the predicted crash frequency for a signalized intersection is 2.33 times higher than that of a TWSC intersection under the same major- and minor-road AADTs. Although this 2.33 value may not be directly interpreted as a CMF for installing a signal, it primarily reflects that the uncalibrated HSM SPFs predict higher crash frequencies at signalized intersections than at TWSC intersections when no local calibration or adjustment factors are applied. Additionally, Qiao et al. (2019) reported higher predicted crash rates at signalized intersections than at TWSC intersections when evaluating the cost-effectiveness of these two facilities in terms of operations and safety.

Based on the above discussion, the safety impacts of signalizing TWSC intersections along rural expressways may be negative or neutral. While specific sites may observe safety improvement from signalization, as demonstrated by earlier studies. Therefore, signalization should be considered only as a last resort for at-grade intersections after a thorough evaluation of other at-grade alternatives, such as roundabouts and RCUTs.

Chapter 7 Operational and Safety Studies of Grade Separation

Transportation agencies may consider grade separation at intersections experiencing substantial safety or operational deficiencies. Grade separation is typically regarded as a last resort because it involves significantly higher construction and right-of-way costs than at-grade alternatives. In such conversions, the major roadway (e.g., expressways or freeways) usually receives priority flow, while cross-street movements are accommodated via ramps and an overpass or underpass. These ramps enable vehicles from the major approach to navigate to the cross-street and vehicles from the cross-street to access the major roadway via on- and off-ramp connections.

This chapter evaluated the operational and safety implications of converting an at-grade intersection, such as a TWSC intersection, into a grade-separated facility, such as a DIstop. The analysis focuses on estimating the expected changes in operational performance and impacts on crash frequency associated with this type of conversion.

7.1 Operational Studies of Grade Separation

Chapters 3 and 4 presented detailed operational analyses of TWSC, signalized, and roundabout intersections, as well as diamond interchange (DIstop) configurations. Building upon those findings, this chapter focuses on evaluating the implications of converting a TWSC at-grade intersection into a grade-separated DIstop facility. To achieve this, the research team analyzed thousands of microsimulation scenarios developed in VISSIM. Traffic volume conditions were adjusted to reflect the operational characteristics of grade separation. For the TWSC cases, the simulation scenarios were modeled using the calibrated parameters and methodologies described in Chapter 3.

Traffic scenarios for the TWSC intersection are provided below.

- Major-road AADT over seven levels (1,500; 2,500; 5,000; 10,000; 15,000; 20,000; 25,000),
- Minor-road AADT over four levels (500; 2,500; 5,000; 7,500),
- Turning movement distributions (left/through/right), in percentages 20/70/10 for major approach and 40/40/20 for minor approaches, and
- Time-of-day (TOD) for four periods:
    - Peak: two 2-hour windows (e.g., 7–9 am and 4–6 pm),
    - Midday peak (e.g., 11am-1pm),
    - Off-peak daytime (Off-Peak 1): all remaining hours between 6 am-midnight, and
    - Off-peak nighttime (Off-Peak 2): midnight–6 am.

When the TWSC intersection is converted into DIstop, left- and through-movements from the major road use the exit ramps. Therefore, 30% of traffic from the major road, in each direction, uses the exit ramps to complete their left- and right-turn movements onto the minor road. On the other hand, 60% of traffic from the minor road in each direction uses entry ramps to join the expressways to complete their corresponding left- and right-turn movements. Therefore, the following traffic scenarios are found for DIstop, when converted from TWSC.

- Major-road AADT over seven levels (1,500; 2,500; 5,000; 10,000; 15,000; 20,000; 25,000),
    - 30% of major-road traffic enters the exit ramps,
- Minor-road AADT over four levels (500; 2,500; 5,000; 7,500),
    - 60% minor-road traffic uses the entry ramps,
- Time-of-day (TOD) for four periods:

- Peak: two 2-hour windows (e.g., 7–9 am and 4–6 pm),
- Midday peak (e.g., 11 am-1 pm),

- Off-peak daytime (Off-Peak 1): all remaining hours between 6 am-midnight, and
- Off-peak nighttime (Off-Peak 2): midnight–6 am.

For the DIstop configuration, a total of 96 traffic scenarios were developed and simulated in VISSIM. Each scenario was replicated 10 times to account for random variation in simulation outcomes, yielding 960 total simulation runs. Delay outputs from these runs were used to evaluate both stopped delay and overall vehicular delay. The methods applied to determine LOS were consistent with the procedures described in Chapters 3 and 4.

Table 7.1 presents the LOS results obtained from the TWSC intersection and DIstop for peak-hour periods.

Table 7.1 LOS comparisons for different traffic scenarios for TWSC intersection and DIstop for peak hour periods

AADT (Major Approach)	AADT (Minor Approach)	LOS (TWSC)	LOS (DIstop)
1,500	5,00	A	A
1,500	2,500	A	A
1,500	5,000	A	A
1,500	7,500	B	A
2,500	5,00	A	A
2,500	2,500	A	A
2,500	5,000	A	A
2,500	7,500	B	A
5,000	5,00	A	A
5,000	2,500	A	A
5,000	5,000	B	A
5,000	7,500	C	A
10,000	5,00	A	A
10,000	2,500	A	A
10,000	5,000	D	A
10,000	7,500	F	A
15,000	5,00	A	A
15,000	2,500	C	A
15,000	5,000	F	A
15,000	7,500	F	A
20,000	5,00	A	A
20,000	2,500	F	A
20,000	5,000	F	A
20,000	7,500	F	A
25,000	5,00	C	A
25,000	2,500	F	A
25,000	5,000	F	A
25,000	7,500	F	C

Table 7.1 shows a clear operational advantage for the DIstop across all traffic conditions analyzed. For lower major-road volumes (up to 10,000 AADT), TWSC intersections generally maintained acceptable performance, with LOS ranging from A to D depending on minor-road demand. However, as the major-road AADT exceeds 10,000, TWSC performance declines

sharply, reaching LOS E or F at higher minor-road volumes. In contrast, the DIstop configuration, benefiting from grade separation, consistently maintains LOS A across most volume combinations and only drops to LOS C under the heaviest traffic scenario (major-road AADT of 25,000 and minor-road AADT of 7,500). These results indicate that grade separation substantially enhances intersection operations by eliminating crossing conflicts and reducing delays, making DIstop a more effective alternative to TWSC intersections when traffic volumes are high or operational efficiency is a priority. However, converting a TWSC intersection to a DIstop configuration may be cost-prohibitive, and this aspect was further evaluated through a benefit-cost analysis presented in Chapter 8.

7.2 Safety Studies of Grade Separation

Studies examining the safety implications of converting at grade intersections to grade separated interchanges are limited. While SPFs for at-grade intersections, such as TWSC and RCUT, and for grade-separated interchanges, such as DI or DDI, were discussed in Chapter 5, a direct crash-frequency comparison among these facility types using SPFs may not yield reliable CMF estimates. Although CMFs inferred from such comparisons can provide a general indication of safety effects, studies that directly derive CMFs for converting TWSC intersections to grade-separated interchanges using before-after or comparison-group methods provide a more appropriate and reliable basis for assessing feasibility through safety implications.

The HSM published CMFs of 0.58 for all severities, 0.43 for injury crashes, and 0.64 for non-injury crashes, respectively, to convert at-grade intersections into grade-separated interchanges (HSM, 2010). HSM listed the CMFs shown in Table 7.2, based on the study by Elvik and Erke (2007).

Table 7.2 Potential crash effects of converting an at-grade intersection into a grade-separated interchange (HSM, 2010; Elvik and Erke, 2007)

Treatment	Setting (Intersection Type)	Crash Type (Severity)	CMF (Std. Error)
Convert at-grade intersection into grade-separated interchange	Setting unspecified (Four-leg intersection, traffic control unspecified)	All crashes in the area of the intersection (All severities)	**0.58 (0.1)**
		All crashes in the area of the intersection (Injury)	**0.43 (0.05)**
		All crashes in the area of the intersection (Non-injury)	**0.64 (0.1)**
	Setting unspecified (Three-leg intersection, traffic control unspecified)	All crashes in the area of the intersection (All severities)	*0.84 (0.2)*
	Setting unspecified (Three-leg or four-leg, signalized intersection)	All crashes in the area of the intersection (All severities)	**0.73 (0.08)**
		All crashes in the area of the intersection (Injury)	**0.72 (0.1)**

NOTE: Traffic Volume (Unspecified); Bold text is used for the more statistically reliable CMFs. These CMFs have a standard error of 0.1 or less. Italic text is used for less reliable CMFs. These CMFs have standard errors between 0.2 to 0.3.

CMFs listed in Table 7.2 do not necessarily indicate a DI facility as an interchange, which is our focus. Elvik and Erke (2007) found a CMF of 0.93 (across all severities), particularly for DI, which is not listed in the HSM but is included in the CMF Clearinghouse (2025). A recent study supported by the North Carolina DOT, directly addressing the CMFs for converting at-grade intersections (i.e., stop-controlled minor approaches and signalized intersections) to DI (Srinivasan et al., 2024), mentioned that the Elvik et al. (2009)'s study (based on the Elvik and Erke's (2007) work) was based on limited sites from Europe, and the reliability and usefulness of those CMFs to the United States context are unknown.

Srinivasan et al. (2024) conducted an empirical Bayes before-after study of 20 at-grade intersections in Minnesota and North Carolina that were previously converted to DIs. The combined results from two states and 20 sites indicated that total crashes decreased by 8%, fatal and injury crashes decreased by 30%, and PDO crashes increased by 11%. These corresponding decreases/increases in crashes due to the conversion from at-grade intersections to DIs correspond to CMFs of 0.92, 0.70, and 1.11 for total crashes, fatal crashes, and injury crashes, respectively.

Given that Elvik and Erke (2007) and Srinivasan et al. (2024) reported similar CMFs of 0.93 and 0.92, respectively, the research team has applied a CMF of 0.92 for all crashes when a TWSC/signalized intersection is converted into a DI. Table 7.3 presents estimated annual crash frequencies for intersections converted from TWSC to grade-separated DI.

Table 7.3 Crash frequency estimation of grade-separated DI using CMF

AADT (Major Approach)	AADT (Minor Approach)	Intersection AADT	TWSC Crash Frequency*	Estimated DI Crash Frequency using CMF
1,500	5,00	2,000	0.12	0.11
1,500	2,500	4,000	1.42	1.31
1,500	5,000	6,500	1.74	1.60
1,500	7,500	9,000	2.13	1.96
2,500	5,00	3,000	0.19	0.17
2,500	2,500	5,000	1.54	1.42
2,500	5,000	7,500	1.89	1.74
2,500	7,500	10,000	2.31	2.13
5,000	5,00	5,500	1.60	1.47
5,000	2,500	7,500	1.89	1.74
5,000	5,000	10,000	2.31	2.13
5,000	7,500	12,500	2.84	2.61
10,000	5,00	10,500	2.41	2.22
10,000	2,500	12,500	2.84	2.61
10,000	5,000	15,000	3.48	3.20
10,000	7,500	17,500	4.28	3.94
15,000	5,00	15,500	3.63	3.34
15,000	2,500	17,500	4.28	3.94
15,000	5,000	20,000	2.45	2.25
15,000	7,500	22,500	2.94	2.70
20,000	5,00	20,500	1.12	1.03
20,000	2,500	22,500	2.30	2.12
20,000	5,000	25,000	3.13	2.88
20,000	7,500	27,500	3.75	3.45
25,000	5,00	25,500	1.35	1.24
25,000	2,500	27,500	2.77	2.55
25,000	5,000	30,000	3.78	3.48
25,000	7,500	32,500	4.54	4.18

Note: *underlined values represent HSM-based crash frequency for TWSC intersection

It is important to note that the annual crash frequencies presented in Table 7.3 serve as the basis for comparing relative safety performance and estimating economic safety benefits, which are incorporated into the benefit-cost analysis for retrofit decision-making discussed later in this report.

Chapter 8 Benefit-Cost Analysis

Improving existing intersections or interchanges, whether through retrofit or complete construction, can potentially yield notable operational and safety benefits. However, the associated financial implications remain a critical component of such decision-making processes. A benefit–cost analysis provides a systematic approach for evaluating whether the monetary value of expected benefits outweighs the costs of implementing and maintaining a proposed improvement.

This chapter outlined the framework used to quantify and compare construction or retrofit costs, and operational and safety benefits. The integration of these factors allows for a comprehensive economic evaluation of alternative designs. The benefit-cost results serve as an essential tool for determining the feasibility and justification of potential improvements and for supporting informed, cost-effective decision-making.

8.1 Costs of Intersections, Interchanges, and Grade Separations

At rural expressways, as part of at-grade intersection conversion decisions, a TWSC intersection may be upgraded to a roundabout, a signalized intersection, or an RCUT configuration. In these cases, construction costs are a key input in the decision-making process. Similarly, for grade-separated facilities, a DIstop or DIsig may be converted into a DDI. In some cases, agencies may also consider costs of grade separation when evaluating the feasibility of converting at-grade intersections to interchange options. Overall, the retrofit or conversion costs of these alternatives play a critical role in the benefit-cost analysis, ensuring that decisions reflect a balanced evaluation of operational performance, safety improvement, and economic feasibility.

8.1.1 Roundabout

Converting a TWSC intersection to a roundabout involves multiple construction elements that can vary significantly based on site conditions, leading to wide cost variations. The extent of earthwork and grading depends on terrain and drainage requirements, especially if major reshaping is needed to accommodate the roundabout's circular geometry. Pavement construction costs can fluctuate based on the size of the roundabout (single-lane vs. multi-lane), turning-lane modifications, truck apron requirements, and the depth of pavement layers required for local traffic volumes. Additional costs arise when curbing, splitter islands, and medians must be heavily reinforced or customized for safety or aesthetic considerations. Drainage system complexity, such as the need for new culverts or stormwater controls, can further impact the budget. Signage, striping, and lighting requirements also vary depending on visibility standards and traffic volumes. Crucially, right-of-way acquisition can greatly increase costs if surrounding land must be purchased or if the intersection is in a constrained corridor. Utility relocation may add another layer of cost variability based on the presence and depth of water, power, or telecom lines. Altogether, these factors make each roundabout project highly site-specific, resulting in a wide range of construction costs.

The FHWA, in a 2000 report, estimated the average cost of 14 roundabouts at around $250,000 each (Robinson et al., 2000). Adjusting for inflation, the estimated construction cost would be around $470,000 in 2025. Table 8.1 lists examples of recently proposed costs for a few roundabout constructions in different states, ranging from $2.4 million to $9.0 million.

Table 8.1 Examples of estimated costs for proposed roundabouts in recent years

State	Location (County)	Publish Year	Estimated Costs
Washington	US 12/Old Hwy 9 (Thurston)	2025	$4.3 million
Washington	SR 3/Mason Lake Rd (Mason)	2025	$5.9 million
Oregon	OR 62/OR 234 (Jackson)	2025	$8.2 million
Oregon	OR 6/NW Aerts Rd (Washington)	2024	$6.0 million
California	4th St/6th Ave (Merced)	2024	$2.7 million
California	Duncan Rd/Comstock Rd (San Joaquin)	2023	$2.4 million
Colorado	US 160/CR 225 (La Plata)	2023	$9.0 million
Michigan	County Rd 492/M 35 (Marquette)	2024	$7.2 million
Georgia	SR 30/Kolic Helmey Rd (Effingham)	2025	$2.9 million

Khattak et al. (2014) compiled construction cost information for roundabouts across multiple states. Table 8.2 presents a selected subset of those roundabouts for which construction year data were available. The reported costs have been updated to 2025 dollars, using two cost categories: actual cost (A) and estimated cost (E). This adjustment allows for consistent comparison of roundabout construction expenditures across different locations and time periods. Table 8.2 shows that the roundabout construction costs ranged from $400,000 to $6.2 million in 2025 currency.

Table 8.2 Examples of roundabout costs from earlier projects

State	Construction Year	Construction Costs (Method)	Costs Adjusted to Year 2025
Kansas	2006	$3.2 million (A)	$5.2 million
Kansas	2001	$2.5 million (A)	$4.6 million
Washington	2014	$4.5 million (E)	$6.2 million
Maryland	1999	$232,284 (A)	$452,486
Maryland	2000	$520,613 (A)	$981,970
Maryland	2001	$300,000 (E)	$550,197
Maryland	2001	$679,569 (E)	$1.2 million
Maryland	1996	$464,540 (A)	$961,648
Maryland	1993	$200,000 (A)	$449,550
Maryland	1995	$472,014 (A)	$1.0 million
Maryland	1995	$493,881 (A)	$1.1 million
Maryland	2001	$300,000 (E)	$550,197
Maryland	1995	$386,145 (A)	$822,965
Maryland	1998	$506,678 (A)	$1.0 million
Maryland	1999	$382,347 (A)	$745,416
Michigan	2013	$2.3 million (E)	$3.2 million
Alabama	2014	$1.4 million (E)	$1.9 million

NDOT reported construction costs of approximately $6.5 million for the multi-lane roundabout built at the intersection of Highway-77 and Highwy-109 (Road 16) in Wahoo, Nebraska. A comparable multi-lane roundabout constructed at 37th Street and Norfolk Avenue in Norfolk, Nebraska incurred a similar cost of around $6.5 million**.** In contrast, the single-lane roundabout near Hastings, Nebraska, at the junction of US-6 and Adams Central Avenue, was completed at a lower cost of approximately $3.7 million. These cost variations primarily reflect differences in lane configuration and right-of-way requirements at each site.

In summary, the cost of a roundabout can vary widely from $250,000 to $7 million, depending on site characteristics and facility needs.

8.1.2 RCUT

Construction of a rural, unsignalized RCUT primarily involves reconstructing the median, adding U-turn crossovers, and installing new turn lanes. Typical work includes widening or rebuilding the median (from approximately 40 to 60 feet for truck accommodation, with loon bulb-outs if needed) and paving new right-turn and deceleration lanes on both the major and minor approach. Earthwork and pavement components (e.g., subgrade, base, surface, and curbing) represent the largest share of costs. The project also includes installing traffic control devices, such as one-way and wrong-way signs, directional arrows, and pavement markings. Right-of-way acquisition and utility relocation often contribute substantially to total cost, along with drainage and illumination improvements that are generally comparable to those for conventional intersections. Retrofit costs for RCUTs largely depend on the extent of modifications required to convert the existing intersection to the RCUT configuration.

Construction cost estimates for RCUT projects in North Carolina were documented by Sangster and Adams (2019). Table 8.3 lists a selected set of those projects, incorporating updated completion years and adjusted 2025 equivalent construction costs.

Table 8.3 Estimated construction costs of RCUTs (revised from Sangster and Adams, 2019)

Location	Number of RCUT	Year Completed*	Construction Costs	Construction Costs (2025)*
NC-24 at Hubert Blvd/Waterfront	1	2014	$1,001,700	$1,390,900
US-17 and Dawson Cabin Road	1	2011	$486,080	$716,900
NC-87 at H.M. Cagle Drive	1	2011	$1,310,400	$1,932,670
US-74 at Old Pageland-Monroe Road	2	2019	$1,186,000	$1,530,400
US-17 at Kelhum Loop Road and Halltown Road	2	2018	$398,750	$522,520
NC-55 Bypass at Avent Ferry Road	1	2016	$1,291,500	$1,770,600
US-17 and Thomasboro Road/Pea Landing Rd	1	2024	$998,000	-
US-64 at Brown's Crossroads	1	2017	$636,540	$851,400
NC-24/27 and Newt Road	1	2023	$675,000	-
US-264 near Neck Road	1	2025	$1,435,000	-
US-401 at North Parker Church Rd and Pitman Grove Church Rd	2	2024	$835,000	-
US-17 and Hickman Rd/SW Middleton Avenue	1	2024	$1,760,000	-

*Completion years have been verified and updated from the original report and projects completed in 2021 or earlier have been adjusted to reflect 2025 equivalent construction costs

Table 8.3 shows that RCUT sample costs in North Carolina ranged from around 0.5 million to $2 million. On the other hand, Ozguven et al. (2019) reported RCUT cost estimates from a survey sent to various state DOTs, as shown in Table 8.4, where costs ranged from $200,000 to $1.3 million.

Table 8.4 RCUT costs from statewide survey (Ozguven et al., 2019)

States	Number of RCUTs	Cost Estimate
Indiana	3	$400,000 (1 site) & $1,200,000 (other 2 sites each)
Mississippi	8	$1,870,000 (on average)
Missouri	19	$650,000 to $700,000
North Carolina	118	$200,000 to $1,300,000
South Carolina	3	$750,000, $810,000, $325,000

Sun et al. (2019) reported that engineers from the Louisiana Department of Transportation and Development (DOTD) estimated an average construction cost of approximately $300,000 for RCUT installations across the state. The Minnesota DOT (2018) reported a higher average cost of around $750,000 per RCUT. In comparison, the Indiana DOT (2025) indicated that retrofitting a conventional intersection to an RCUT or a similar configuration can range from a few hundred thousand dollars to more than $1 million, depending on site conditions and design complexity.

In the state of Nebraska, the three RCUTs located at Murray, North Bend, and Humphrey cost around $1.7 million, $1.7 million, and $3 million, respectively.

In summary, RCUT construction costs generally range from $200,000 to about $4 million, depending on site characteristics, design complexity, and the extent of required roadway modifications.

8.1.3 Signalized Intersection

While rural intersections are typically managed effectively with unsignalized controls, certain locations, particularly those with higher volumes or near suburban development, may require signalization to ensure smooth, efficient, and safe traffic operations. The cost of converting a rural TWSC intersection to a signalized configuration can vary significantly based on site conditions and design scope. While some low-volume locations may only require signal hardware and minor pavement work, most rural conversions involve added turn lanes, taper extensions, and geometric realignments as the signalized layout may differ substantially from the original TWSC configuration. Signalization-related elements often drive much of the project cost, including mast arms, poles, signal heads, controller cabinets, detection systems, and electrical conduit. Construction of turning lanes is often a substantial cost. Additional costs may arise from signing and striping changes, pedestrian features such as crosswalks and push-button signals, and potential utility relocations or right-of-way acquisition needed to accommodate expanded infrastructure. Like many other DOTs, NDOT requires intersection lighting for all signalized locations; as a result, lighting costs are added when upgrading previously unlit intersections. Overall, signal system installation and geometric modifications account for the bulk of the construction costs to upgrade a TWSC to signalized intersection options.

Several recent examples highlight the range of costs associated with converting TWSC intersections to signalized intersections. In 2025, a full mast-arm traffic signal was proposed at the rural intersection of US-98 and County Road 91 in Lillian, Baldwin County, Alabama, with an estimated total cost of approximately \$1 million. That same year, the California Department of Transportation (Caltrans) initiated a safety improvement project to install a traffic signal at the TWSC intersection of County Road P and SR 32 in Plumas County, with a projected cost of approximately \$4.8 million. Also in 2025, the Michigan DOT planned to install signals at a rural three-legged stop-controlled intersection at M-26 and Canal Road in Houghton, with an estimated cost of \$194,000. In contrast, a 2025 project by the Mississippi DOT at the three-legged intersection of State Route 28 and State Route 37 in Taylorsville was estimated at $5.5 million, including intersection realignment and geometric improvements, in addition to signalization.

NDOT has reported varying costs for converting TWSC intersections to signalized intersections, depending on geometric features and the scope of improvements. In 2022, a four-legged TWSC intersection at the junction of N-50 and Platteview Road in Springfield, Nebraska, was converted to a signalized intersection with associated turn lane construction at an approximate cost of $2 million. In contrast, a 2014 project at the four-legged intersection of US-6 and Harrison Street, north of Gretna, involved only signalization, without turn-lane additions, at a cost of approximately $150,000, which equates to roughly $200,000 in 2025 dollars. Additionally, a three-legged TWSC intersection at US-30 and Industrial Drive in Blair was converted to a signalized intersection with a minor left-turn lane in 2021 at a reported cost of approximately $500,000.

In summary, the cost of converting a TWSC intersection to a signalized intersection can range from $150,000 for basic signal installation to $5 million when realignment or other roadway improvements are included in the project scope.

8.1.4 Diamond and Diverging Diamond Interchange

Transportation agencies often convert existing DIstop configurations to DIsig or DDI, or DIsig to DDI, to address increasing traffic demand and enhance safety. Compared to intersection conversions at the same grade, modifying interchanges typically involves higher costs due to the need for modification in larger footprints, elevated designs, and more complex geometric and traffic flow considerations.

DDI offers a notable cost advantage over other typical interchange designs, making it increasingly popular in recent years (Cunningham et al., 2021). One key benefit is its ability to leverage existing infrastructure, such as bridges and rights-of-way, which can significantly reduce design and construction costs and timelines. This makes the DDI a practical option, especially when retrofitting existing interchanges.

As discussed, structural requirements primarily influence interchange construction costs, and a DDI’s ability to minimize new structural needs makes it particularly cost-effective. However, actual project costs vary depending on site-specific conditions and whether retrofit or new constructions are required (Cunningham et al., 2021). Table 8.5 presents examples of retrofit and newly constructed DDIs completed in different states across various years.

Table 8.5 New construction or retrofit costs of DDIs in different states

State	Interchange Location	Open to Traffic	Retrofit	Construction Cost	Construction Cost (2025)*
Missouri	MO-13 and I-44 (Springfield)	2009	Yes	\$3.2 million	\$4.9 million
Tennessee	Bessemer St. and US-129 (Alcoa)	2010	Yes	\$2.9 million	\$4.4 million
Utah	Timpanogos Hwy and I-15 (Lehi)	2011	Yes	\$8.5 million	\$12.3 million
New York	Winton Rd. and I-590 (Rochester)	2012	Yes	\$4.5 million	\$6.5 million
Missouri	National Ave. and US-60 (Springfield)	2012	Yes	\$8.2 million	\$11.8 million
Wyoming	I-25 & College Dr .(Casper)	2015	Yes	\$3.1 million	\$4.3 million
Florida	I-75 and SR 56 (Lutz/Wesley Chapel)	2023	Yes	\$34.8 million	-
Kansas	K-10 & 6th St (Lawrence)	2024	Yes	\$14 million	-
Iowa	I-80 and 1st Avenue (Coralville)	2025	Yes	\$35.6 million	-
Ohio	SR 46 & SR 82 (Howland Township)	2025	Yes	\$20 million	-
Utah	Pioneer Crossing and I-15 (American Fork)	2010	No	\$22 million	\$32.9 million
Missouri	Mid Rivers and I-70 (St. Peters)	2013	No	\$14 million	\$19.8 million
Minnesota	CR-120 and Hwy 15 (St. Cloud)	2013	No	\$17.5 million	\$24.6 million
Arizona	I-10 & Houghton Rd (Tucson)	2014	No	\$24.4 million	\$33.9 million
Missouri	I-55 & US-61 (Cape Girardeau)	2021	No	\$17.5 million	-

*Interchange constructed before 2021 are considered to estimate for 2025 equivalent costs

Table 8.5 illustrates that Retrofit DDIs, those built by modifying existing conventional interchanges, typically exhibit lower costs, with adjusted 2025 construction values ranging from approximately \$4.3 million to \$12.3 million. However, constructions actually built or proposed after 2021 cost relatively more, ranging from \$14 million to \$34.8 million. The average costs of these retrofit samples are around $14.9 million. Examples include retrofit projects such as I-25 and College Drive in Casper, Wyoming ($4.3 million) and I-80 and 1st Avenue in Coralville, Iowa ($35.6 million). In contrast, new DDI constructions, which require full interchange replacement or new infrastructure, tend to be significantly more expensive, on average. Reported costs for these projects range from about $19.8 million to over $33.9 million, with recent large-scale examples such as I-10 and Houghton Road in Tucson, Arizona ($33.9 million, 2025 equivalent) and CR-120 and Highway 15 in St. Cloud, Minnesota ($24.6 million). The average cost of the newly constructed DDI samples is around $25.8 million. Overall, the data indicate that retrofit DDIs generally fall below $15 million. In contrast, new DDI constructions can exceed $30 million, reflecting the greater scope of structural, right-of-way, and traffic control improvements required for new builds.

NDOT reported a project cost of over $35 million for DDI constructed around 2016 at I-80 and NW 48th Street near Lincoln, Nebraska. The high cost of this project was primarily due to extensive lane reconstruction and the construction of two bridges, which significantly increased overall costs. More recently, NDOT estimated a cost of approximately $17 million for the DDI at US-6 and 192nd Street in Omaha, Nebraska, reflecting a more typical range for DDI construction replacing a conventional diamond interchange.

In summary, construction costs for retrofit DDIs typically range from $4 million to $15 million, while new DDI construction costs are generally higher, averaging $20 million to $35 million. Recent projects indicate an upward trend in construction costs for post-2021 DDI implementations. Overall, the cost variation reflects differences in project scope, structural requirements, and site conditions.

8.1.5 Grade Separation

Converting an at-grade intersection to a grade separated interchange is generally more complex and costly than retrofitting either an at-grade intersection or an existing grade separated interchange. This is because it involves major construction components, such as bridges, elevation changes, and new ramp systems, as well as potential right-of-way acquisition and utility relocations. Due to the scale and complexity of new grade separation projects, transportation agencies typically reserve them for locations with significant operational or safety needs where simpler improvements are insufficient.

State transportation agencies and prior studies indicate a wide range of costs, depending on project scope and site characteristics. Converting an at-grade intersection to a diamond interchange is a major investment, often exceeding $10 million (Srinivasan et al., 2024). According to the New Jersey DOT (2024) cost guidelines, replacing an at-grade intersection with a grade-separated interchange typically ranges from $18.6 million to $36.3 million, with a reported median cost of $34.6 million**.** Likewise, planning studies conducted by the Metropolitan Council and Minnesota DOT (2020) identified potential interchange improvements ranging from less than $10 million to more than $100 million across 56 sites in the Twin Cities metropolitan area.

Recently, in Indiana, a project along the US 31 corridor was estimated at approximately $50 million, reflecting a large-scale rural interchange conversion with modern design standards and substantial supporting infrastructure (Indiana DOT, 2025). This higher-end figure underscores the potential cost of comprehensive upgrades that may include long bridges, extensive ramps, and full access control. In contrast, Iowa’s US 30 corridor expansion plan used a legislative estimate of about $16 to $17 million per new interchange (Iowa House, 2024).

These values were based on planning-level assumptions for typical rural expressway interchanges and are likely to reflect more modest designs than the Indiana example. NDOT reported an estimated cost of around $28 million for constructing the interchange at US-77/Warlick Blvd in Lincoln, Nebraska, which is relatively mid-range.

In summary, developing a grade-separated interchange facility is a costly undertaking, typically ranging from $10 million to $50 million, with the final cost determined mainly by site-specific conditions and design requirements.

8.2 Costs of Traffic Delays and Crashes

Quantification of the operational costs of vehicle movements around intersections and interchanges requires two major components: i) value of time, and ii) idling cost. The unit value of time is used to quantify the costs of delay or travel time savings. Idling costs occur when vehicles are stopped entirely at intersections and interchanges.

Based on the literature review of operational cost components for National use and Nebraska-based studies (AASHTO 2010; Tufuor et al., 2022) and the conversion of prices from the previous year, the cost categories applied are shown in Table 8.6.

Table 8.6 Operational cost components

Cost Component	**Vehicle Composition**		
	Passenger car	**Single-unit truck**	**Tractor-trailer**
Value of time	29.18 ($/hr)	31.55 ($/hr)	33.45 ($/hr)
Idling cost	1.00 ($/hr)	2.50 ($/hr)	3.50 ($/hr)

NDOT provided the latest societal crash costs (i.e., comprehensive crash costs that sum direct economic costs and the indirect costs of loss of quality of life) for Nebraska (NDOT HSIP, 2025; NDOT Crash Costs, 2023). This method is used nationally for benefit-cost analysis of crash impacts (Harmon et al., 2018). Table 8.7 lists the societal crash costs in Nebraska.

Table 8.7 Societal crash costs in Nebraska

Crash Severity	Cost Per Crash
K (fatal injury)	$19,244,830
A (suspected serious injury)	$1,115,980
B (suspected minor injury)	$338,200
C (possible injury)	$213,990
PDO (property damage-only)	$20,280

8.3 Benefit-Cost Analysis Framework

The goal of the operational benefit-cost analysis was to conduct an economic feasibility analysis of converting one intersection type to another, considering the operational benefits and construction/retrofit costs associated with the conversion. The operational benefit-cost framework is shown using **Equations 8.1, 8.2, and 8.3**. The framework is illustrated using the example of the operational feasibility analysis of Models 1 (TWSC vs Roundabout), 2 (TWSC vs RCUT), and 3 (RCUT vs Roundabout) discussed in Chapter 3.

$$\mathrm{OB}_{\mathrm{day,M}} = \sum_{i=1}^{4}\sum_{j=1}^{3}[\mathrm{V}_{\mathrm{M,j,TOD_i}} \times ((\mathrm{VOT_j} \times (\mathrm{D}^{\mathrm{base}}_{\mathrm{M,j,TOD_i}} - \mathrm{D}^{\mathrm{conv}}_{\mathrm{M,j,TOD_i}})) + (\mathrm{IC_j} \times (\mathrm{d}^{\mathrm{base}}_{\mathrm{M,j,TOD_i}} - \mathrm{d}^{\mathrm{conv}}_{\mathrm{M,j,TOD_i}})))] \quad (8.1)$$

$$\mathrm{OB}_{\mathrm{year,M}} = 365 \times \mathrm{OB}_{\mathrm{day,M}} \quad (8.2)$$

$$OB_{NPV,M} = OB_{year,M} \times P_{M,A,r,n}$$

$$OB_{NPV,M} = OB_{year,M} \times [\frac{(1+r)^n - 1}{r \times (1+r)^n}] \quad (8.3)$$

M = Model 1, or, 2, or 3

$OB_{day,M}$ = Operational benefit for given model for a day

$OB_{year,M}$ = Operational benefit for given model for a year

i =1, 2, 3, 4: Index for TOD

TOD1 = TOD_PEAK (for 4 hours)

TOD2 = TOD_MID (for 2 hours)

TOD3= TOD_OFF1 (for 12 hours)

TOD4=TOD_OFF2 (for 6 hours)

j =1, 2, 3: Index for vehicle mode for PC, single unit truck, and tractor-trailer truck, respectively

V_{M,j,TOD_i} = Number of vehicles of mode j during TOD_i for model M

VOTj = Mode j user's time value (dollars/sec)

ICj = Idling cost for mode j (dollars/sec)

D_{M,j,TOD_i}^{base} = Average vehicular delay for base condition for Model M (sec/veh) (e.g., TWSC is base condition for Model 1)

D_{M,j,TOD_i}^{conv} = Average vehicular delay for conversion condition for Model M (sec/veh) (e.g., Roundabout is conversion condition for Model 1)

d_{M,j,TOD_i}^{base} = Average stopped delay for base condition for Model M (sec/veh)

d_{M,j,TOD_i}^{conv} = Average stopped delay for conversion condition for Model M (sec/veh)

$P_{M,A,r,n}$ = Factor for Single Present Value for given rate of return (r%) and design year (n) for Model M

$OB_{NPV,M}$ = Operational benefit in terms of Net Present Value for Model M (dollars)

On the other hand, the safety benefit-cost framework is illustrated using Equations (8.4) and (8.5).

$$SB_{year,M} = \sum_{i=1}^{5}\left(\left(CF_{M,i}^{base} \times CC_{M,i}^{base}\right) - \left(CF_{M,i}^{conv} \times CC_{M,i}^{conv}\right)\right) \tag{8.4}$$

$$SB_{NPV,M} = SB_{year,M} \times P_{M,A,r,n}$$

$$SB_{NPV,M} = SB_{year,M} \times \left[\frac{(1+r)^n - 1}{r \times (1+r)^n}\right] \tag{8.5}$$

Where,

$SB_{year,M}$ = Safety benefit for given model for a year

i =1, 2, 3, 4, 5: Index for crash severity type

1 = K (fatal injury)

2 = A (suspected serious injury)

3= B (suspected minor injury)

4= C (possible injury)

5 = PDO (property damage-only)

$CF_{M,i}^{base}$ = Crash frequency (per year) for i type of severity for base condition for Model M (e.g., TWSC is base condition for Model 1)

$CF_{M,i}^{conv}$ = Crash frequency (per year) for i type of severity for conversion condition for Model M (e.g., Roundabout is conversion condition for Model 1)

$CC_{M,i}^{base}$ = Crash costs for each of i type of severity for base condition for Model M (dollars)

$CC_{M,i}^{conv}$ = Crash costs for each of i type of severity for conversion condition for Model M (dollars)

$P_{M,A,r,n}$ = Factor for Single Present Value for given rate of return (r%) and design year (n) for Model M

$SB_{NPV,M}$ = Safety benefit in terms of Net Present Value for Model M (dollars)

8.4 Benefit-Cost Analysis Based on Operation

8.4.1 Benefit-Cost-Based Intersection Choice Model

Chapter 3 (Section 3.2.3) used logistic regression to build an intersection choice model based solely on operational performance. In other words, when input on AADTs, left-turning percentages from major and minor approaches, and TOD information were provided, the model selected an intersection with lower delay. While this model provided preliminary insights, actual conversions incur significant costs that require justification through benefit-cost analysis. This section applied the logistic regression method to build intersection choice models based on the operational benefit-cost output for a few levels of construction costs.

The operational benefit-cost analysis was conducted across 1,344 traffic scenarios (448 per model, corresponding to AADT and turn proportion) for Models 1, 2, and 3. The goal was to assess whether converting intersection types is economically justified based on operational benefits, and to develop data-driven selection models and guidance.

For the logistic regression model, three conversion cost scenarios of $250,000, $1 million, and $3 million were used. Using a 20-year design life, a benefit-cost analysis was conducted at a 6% discount rate using Equations (8.1) to (8.3). Conversions with a benefit-cost ratio greater than 1.0 were deemed feasible and were used to develop the corresponding logistic regression models.

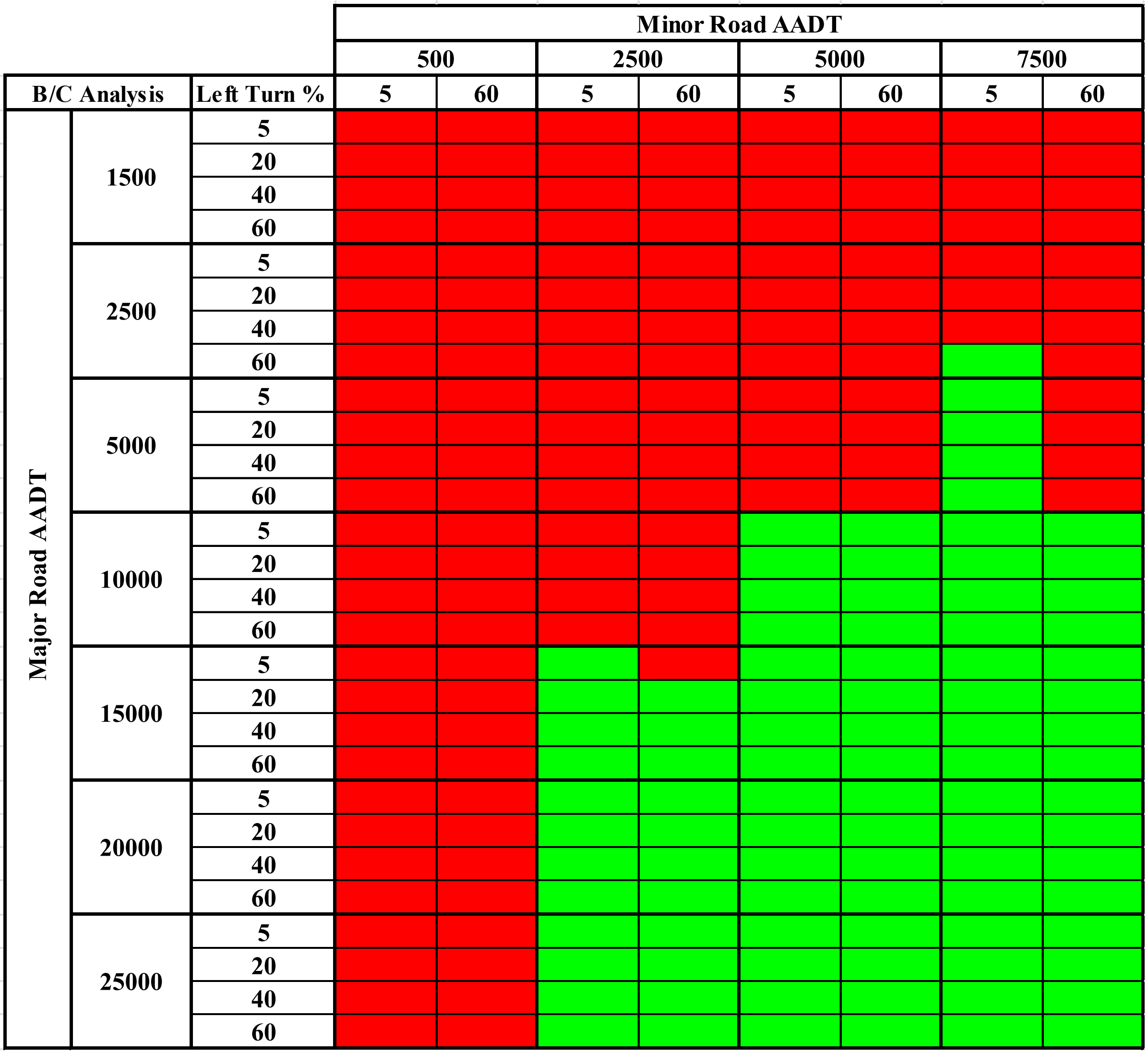


Figure 8.1 Benefit-cost analysis of converting TWSC to roundabout based on cost of $1 million

As an example of benefit-cost analysis results, Figure 8.1 shows a subset of scenarios for the feasibility of converting TWSC to a roundabout (i.e., Model 1) for $1 million, indicating "green" cells as feasible and "red" cells as not feasible.

Table 8.8 lists the logistic regression models for intersection choices based on the three conversion costs for Models 1, 2, and 3.

Table 8.8 Logistic regression models for intersection choice with different conversion costs

Intersection Groups	Parameters	Conversion Cost ($250,000)			Conversion Cost ($1 million)			Conversion Cost ($3 million)		
		Est.	SE	p val.	Est.	SE	p val.	Est.	SE	p val.
Model 1 TWSC vs Roundabout	Intercept	-2.881	0.454	0.000	-5.041	0.583	0.000	-6.475	0.611	0.000
	AADT_M	0.111	0.018	0.000	0.242	0.023	0.000	0.233	0.023	0.000
	AADT_C	0.847	0.077	0.000	0.600	0.070	0.000	0.623	0.073	0.000
	LT_M	-0.013	0.006	0.046	-0.015	0.006	0.020	-	-	-
	LT_C	-0.011	0.006	0.076	-0.010	0.006	0.131	-	-	-
	AIC		345.277			337.414			316.53	
	McFadden R^2		0.444			0.456			0.449	
	Cox–Snell R^2		0.450			0.458			0.432	
	Nagelkerke R^2		0.608			0.619			0.603	
	Cor. Pred. %		87.280			86.160			88.170	
Model 2 TWSC vs RCUT	Intercept	-7.090	0.759	0.000	-7.422	0.797	0.000	-7.708	0.809	0.000
	AADT_M	0.326	0.029	0.000	0.335	0.030	0.000	0.309	0.029	0.000
	AADT_C	0.453	0.070	0.000	0.504	0.007	0.003	0.560	0.076	0.001
	LT_M	0.028	0.007	0.0003	0.0003	0.007	0.003	0.024	0.007	0.001
	LT_C	0.0006	0.007	0.933	0.0003	0.007	0.962	-	-	-
	AIC		281.576			271.65			281.269	
	McFadden R^2		0.551			0.563			0.535	
	Cox–Snell R^2		0.529			0.529			0.502	
	Nagelkerke R^2		0.709			0.717			0.689	
	Cor. Pred. %		*91.29*			*91.520*			*90.620*	
Model 3 RCUT vs Roundabout	Intercept	1.378	0.416	0.001	0.633	0.446	0.156	-10.170	1.474	0.000
	AADT_M	-0.154	0.020	0.000	-0.186	0.026	0.000	-0.010	0.025	0.666
	AADT_C	0.848	0.084	0.000	1.235	0.121	0.000	2.117	0.289	0.000
	LT_M	-0.056	0.008	0.000	-0.067	0.010	0.000	-0.097	0.016	0.000
	LT_C	-0.001	0.006	0.796	-0.041	0.008	0.000	-0.009	0.010	0.349
	AIC		319.352			249.400			151.589	
	McFadden R^2		0.479			0.614			0.704	
	Cox–Snell R^2		0.471			0.573			0.530	
	Nagelkerke R^2		0.641			0.764			0.806	
	Cor. Pred. %		*82.140*			*87.050*			*92.410*	

Note: Est. (Estimated), SE (Standard Error), p val. (p value), Cor. Pred. % (Correct Prediction %)

Coefficients from Table 8.8 can be used for intersection conversion decisions by applying Equation (3.2).

Chapter 9 presented the conversion guidelines developed from the model outcomes summarized in Table 8.8. Additional guidance was provided for cases where the estimated construction cost exceeds the $3 million threshold. When costs become substantially higher, intersection conversion decisions are likely to be governed primarily by AADT, with reduced sensitivity to turning-movement proportions and a diminished need for detailed modeling. Conversely, when costs are low to moderate, conversion decisions tend to be more complex and may require modeling-based evaluations to support quick and informed decision-making.

8.4.2 Benefit-Cost-Based Interchange Choice Model

Chapter 4 (Section 4.2.2) used log-linear regression to model the comparative interchange delays. In other words, when input on AADTs, left-turning percentages from major and minor approaches, and TOD information was provided, the model produced delay information for different interchanges. This way, by comparing delays across two interchanges, the facility type with better performance could be identified. While this model provided preliminary insights, actual conversions incur significant costs and require justification through an operational benefit-cost analysis.

This section applied the logistic regression method to build interchanges choice model based on the operational benefit-cost output for a few levels of construction costs. A comprehensive pairwise benefit-cost analysis across 1,536 traffic scenarios (512 scenarios per model for corresponding AADT and turn proportion) was conducted for Models 4 (DIstop vs. DIsig), 5 (DIstop vs. DDI), and 6 (DIsig vs. DDI). The goal was to assess whether converting interchange types is economically justified and to develop data-driven selection models and guidance.

Three levels of conversion cost scenarios of $1 million, $5 million, and $10 million were used for Model 4 and Model 5. For Model 6, converting from DIsig to DDI, the three cost ranges were $ 1 million, $ 3 million, and $ 5 million, considering that DDI may be constructed within a similar footprint to the existing DIsig, resulting in relatively lower costs. Note that this potential cost savings is one of the reasons for the wide implementation of DDI (Cunningham et al., 2021).

Using a 20-year design life, benefit-cost analysis was conducted at a 6% discount rate using Equations (8.1) to (8.3). Conversions with a benefit-cost ratio greater than 1.0 were deemed feasible and used to develop the corresponding logistic regression models.

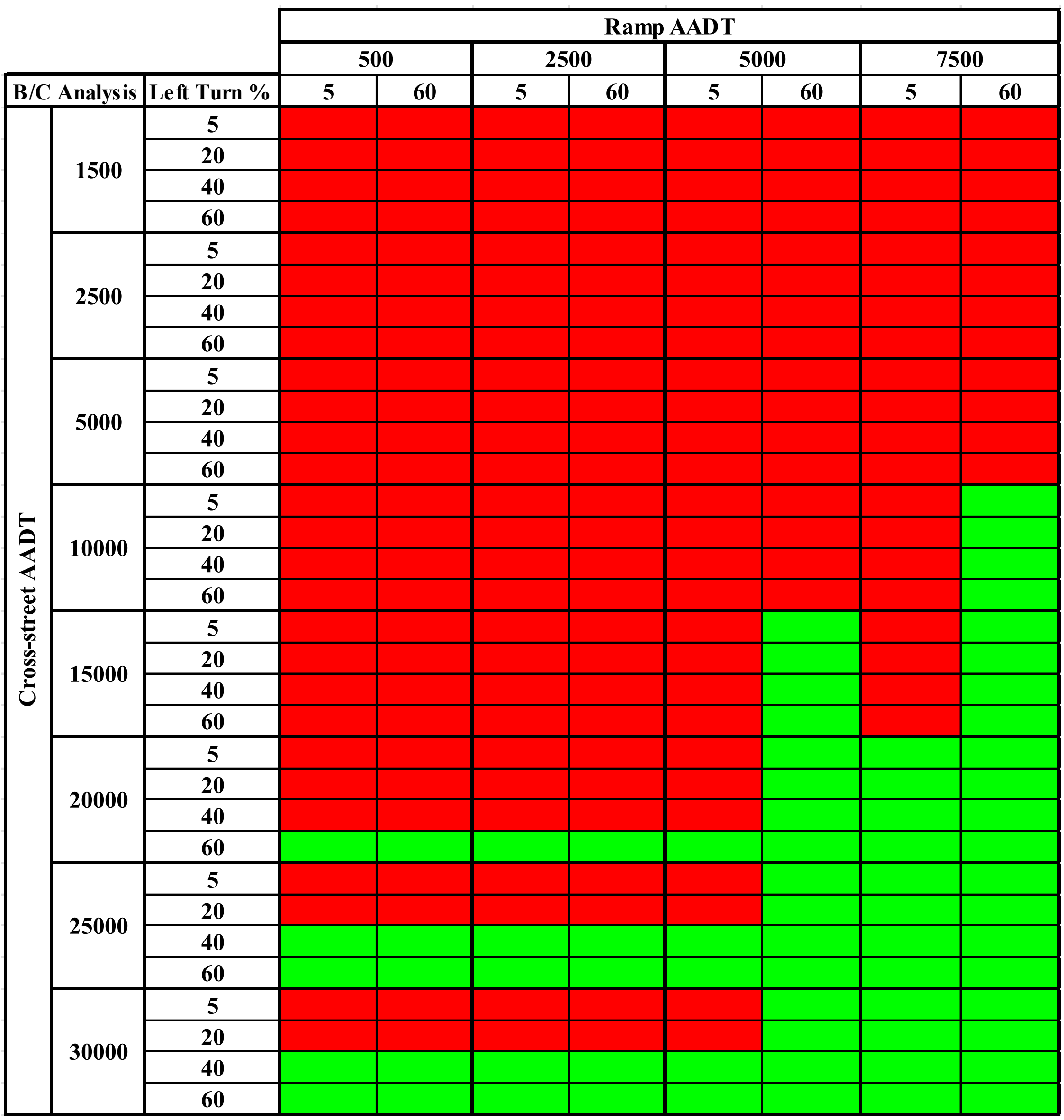


Figure 8.2 Benefit-cost analysis of converting DIstop to DIsig based on cost of $5 million

As an example of benefit-cost analysis results, Figure 8.2 shows a subset of scenarios for the feasibility of converting DIstop to DIsig (i.e., Model 4) for $5 million, indicating "green" cells as feasible and "red" cells as not feasible.

The research team developed a series of binary logistic regression models to examine the likelihood of selecting a given interchange in a pairwise comparison across thousands of scenarios. Logistic regression models the log-odds of a binary outcome (i.e., preferred interchange type based on benefit-cost ratio in this case) as a linear function of predictor variables. Equation (3.2) showed the general form of the model.

Table 8.9 shows the results of the logistic regression for Model 4 (probability of selecting DIsig over DIstop), Model 5 (probability of selecting DDI over DIstop), and Model 6 (probability of selecting DDI over DIsig) based on operational benefit-cost study outcomes.

Table 8.9 Logistic regression models for interchange choice for different conversion costs

Interchange Groups	Parameters	Conversion Cost $1 million (Models 4, 5, & 6)			Conversion Cost $5 million (Models 4 & 5) $3 million (Model 6)			Conversion Cost $10 million (Models 4 & 5) $5 million (Model 6)		
		Est.	**SE**	**p val.**	**Est.**	**SE**	**p val.**	**Est.**	**SE**	**p val.**
Model 4 DIstop vs. DIsig	Intercept	-17.002	1.865	0.000	-16.141	1.740	0.000	-14.342	1.495	0.000
	AADT_C	0.425	0.046	0.000	0.396	0.042	0.000	0.342	0.036	0.000
	AADT_R	0.864	0.114	0.000	0.781	0.105	0.000	0.665	0.091	0.000
	LT_C	0.093	0.013	0.000	0.089	0.012	0.000	0.086	0.011	0.000
	LT_R	0.055	0.013	0.000	0.053	0.010	0.000	0.038	0.009	0.000
	AIC		193.884			202.188			220.945	
	McFadden R^2		0.703			0.683			0.639	
	Cox–Snell R^2		0.574			0.554			0.517	
	Nagelkerke R^2		0.817			0.799			0.760	
	Cor. Pred. %		*91.600*			*91.600*			*90.620*	
Model 5 DIstop vs DDI	Intercept	-19.729	2.278	0.000	-17.783	1.982	0.000	-18.065	2.031	0.000
	AADT_C	0.502	0.057	0.000	0.442	0.049	0.000	0.442	0.049	0.000
	AADT_R	1.052	0.139	0.000	0.881	0.118	0.000	0.863	0.117	0.000
	LT_C	0.116	0.016	0.000	0.099	0.014	0.000	0.103	0.014	0.000
	LT_R	0.063	0.011	0.000	0.060	0.011	0.000	0.058	0.011	0.000
	AIC		171.825			186.762			183.004	
	McFadden R^2		0.748			0.714			0.713	
	Cox–Snell R^2		0.609			0.578			0.569	
	Nagelkerke R^2		0.852			0.824			0.821	
	Cor. Pred. %		*93.360*			*91.990*			*92.380*	
Model 6 DIsig vs. DDI	Intercept	-9.910	0.990	0.000	-11.246	1.097	0.000	*-40.079	7.220	0.000
	AADT_C	0.267	0.027	0.000	0.302	0.028	0.000	1.278	0.220	0.000
	AADT_R	0.950	0.099	0.000	0.320	0.065	0.000	0.219	0.129	0.089
	LT_C	0.093	0.011	0.000	0.089	0.010	0.000	0.271	0.055	0.000
	LT_R	0.037	0.008	0.000	0.038	0.008	0.000	0.028	0.016	0.087
	AIC		275.912			272.149			73.758	
	McFadden R^2		0.609			0.596			0.847	
	Cox–Snell R^2		0.554			0.530			0.670	
	Nagelkerke R^2		0.754			0.738			0.918	
	Cor. Pred. %		*88.670*			*90.23*			*97.5*	

Note: Est. (Estimated), SE (Standard Error), p val. (p value), Cor. Pred. % (Correct Prediction %)
*Model 6 with high cost is applicable for AADT_C of 10,000 and over.

Using the results from Table 8.9, Equation (8.6) shows an example application of logistic regression for moderate cost in Model 5.

$$\log\left(\frac{P_{\text{Model 5}}}{1-P_{\text{Model 5}}}\right) = -17.783 + (0.442\times AADT\_C) + (0.881\times AADT\_R) + (0.099\times LT\_C) - (0.060\times LT\_R) \qquad (8.6)$$

Where,

$P_{\text{Model 5}}$ = probability of choosing a DDI over DIstop

The rest of the variables are defined under Equation (3.2).

The coefficients of AADT_C and AADT_R in Equation (8.6) are positive and measured 0.442 and 0.881, indicating that each unit of AADT increased the odds of selecting DDI over DIstop by 56% and 141%, holding all other variables constant. Similarly, a 1% increase in the percentage of cross-street and ramp left-turns increased the odds of DDI selection by 10% and 6%, respectively.

A predicted probability of 0.5 or higher indicates preference for the corresponding alternate interchange. Using this threshold, Table 8.9 lists the correct prediction rates for all models.

Chapter 9 presented the conversion guidelines developed from the model outcomes summarized in Table 8.9. Similar to the intersection, additional guidance was provided for cases where the estimated construction cost of the interchange exceeds the $10 million threshold.

8.5 Benefit-Cost Analysis Based on Safety

8.5.1 Benefit-Cost Analysis of Intersection Safety

This section discussed the economic feasibility of converting the TWSC intersection into an RCUT and a roundabout.

Table 8.10 shows the annual crash cost savings from implementing RCUT for different AADT combinations and the corresponding benefit-to-cost ratio for retrofit costs ranging from $250,000 to $7 million. The benefit-cost analysis was conducted using Equations (8.4) to (8.5).

Table 8.10 Safety benefit-cost analysis of converting TWSC to RCUT intersections

AADT (Major Approach)	AADT (Minor Approach)	TWSC Crash Frequency (per year)	RCUT Crash Frequency (per year)	Yearly Crash Costs Savings by RCUT	Benefit/Cost Ratio				
					Retrofit Cost in million (m)				
					$0.25 m	$1 m	$3 m	$5 m	$7 m
1,500	5,00	0.12	0.08	$23,643	1.0	0.3	0.1	0.1	0.0
1,500	2,500	1.42	0.93	$279,778	11.8	3.1	1.1	0.6	0.5
1,500	5,000	1.74	1.13	$342,827	14.4	3.8	1.3	0.8	0.6
1,500	7,500	2.13	1.39	$419,667	17.6	4.7	1.6	1.0	0.7
2,500	5,00	0.19	0.12	$37,435	1.6	0.4	0.1	0.1	0.1
2,500	2,500	1.54	1.00	$303,421	12.8	3.4	1.2	0.7	0.5
2,500	5,000	1.89	1.23	$372,381	15.6	4.2	1.4	0.9	0.6
2,500	7,500	2.31	1.51	$455,132	19.1	5.1	1.7	1.0	0.7
5,000	5,00	1.60	1.04	$315,243	13.2	3.5	1.2	0.7	0.5
5,000	2,500	1.89	1.23	$372,381	15.6	4.2	1.4	0.9	0.6
5,000	5,000	2.31	1.51	$455,132	19.1	5.1	1.7	1.0	0.7
5,000	7,500	2.84	1.85	$559,556	23.5	6.3	2.1	1.3	0.9
10,000	5,00	2.41	1.57	$474,835	20.0	5.3	1.8	1.1	0.8
10,000	2,500	2.84	1.85	$559,556	23.5	6.3	2.1	1.3	0.9
10,000	5,000	3.48	2.27	$685,654	28.8	7.7	2.6	1.6	1.1
10,000	7,500	4.28	2.79	$843,275	35.4	9.5	3.2	1.9	1.4
15,000	5,00	3.63	2.37	$715,208	30.1	8.0	2.7	1.6	1.2
15,000	2,500	4.28	2.79	$843,275	35.4	9.5	3.2	1.9	1.4
15,000	5,000	2.45	1.60	$482,716	20.3	5.4	1.8	1.1	0.8
15,000	7,500	2.94	1.92	$579,259	24.3	6.5	2.2	1.3	0.9
20,000	5,00	1.12	0.73	$220,670	9.3	2.5	0.8	0.5	0.4
20,000	2,500	2.30	1.50	$453,162	19.0	5.1	1.7	1.0	0.7
20,000	5,000	3.13	2.04	$616,694	25.9	6.9	2.3	1.4	1.0
20,000	7,500	3.75	2.45	$738,851	31.0	8.3	2.8	1.7	1.2
25,000	5,00	1.35	0.88	$265,986	11.2	3.0	1.0	0.6	0.4
25,000	2,500	2.77	1.81	$545,765	22.9	6.1	2.1	1.2	0.9
25,000	5,000	3.78	2.46	$744,762	31.3	8.4	2.8	1.7	1.2
25,000	7,500	4.54	2.96	$894,502	37.6	10.0	3.4	2.0	1.5

Note: *underlined values represent HSM-based crash frequency for TWSC intersection

Results from Table 8.10 clearly indicate that both the crash-reduction potential and the economic return are highly sensitive to traffic demand levels on the major and minor approaches and to retrofit costs.

At lower traffic volumes, such as a major-road AADT of 1,500 and a minor-road AADT of 500, the annual crash cost savings were relatively modest (about $23,600), and the benefit-cost ratio only approached 1.0 when the retrofit cost was as low as $0.25 million. As traffic volumes increased, both the safety benefits and cost-effectiveness of the RCUT retrofit improved markedly. For example, when the major-road AADT remained 1,500 but the minor-road AADT increased to 7,500, the yearly crash savings rose to nearly $420,000, yielding a benefit-cost ratio of 17.6 at a $0.25 million cost and remaining above 1.0 even at a $5 million retrofit cost. However, it is important to note that while RCUT shows safety benefits in such lower traffic volumes, in terms of operational impacts, the conversion will not be feasible. It is important to observe the benefit-cost ratio when major-road traffic demand approaches 5,000 and beyond, with minor-road AADT close to or lower than the major-road AADT, which aligns with typical rural expressway scenarios.

With a major-road AADT of 10,000 and a minor-road AADT ranging from 500 to 7,500, the estimated crash cost savings ranged from $474,000 to $840,000 per year, corresponding to benefit-cost ratios of 5.3 to 9.5 for a $1 million retrofit and from 1.1 to 1.9 for a $5 million retrofit. Even when retrofit costs reached $7 million, the project maintained a favorable benefit-cost ratio above 1.0, even with relatively higher minor demand. These results demonstrated that the RCUT alternative becomes increasingly economically justified as overall intersection volumes grow.

At very high major-road volumes (15,000–25,000 major-road AADT), the benefits remained strong, with annual crash cost savings between $220,000 and $895,000. For these higher-volume intersections, the RCUT retrofit yielded a benefit-cost ratio exceeding 1.0 for almost all cost scenarios when the minor-road AADT was 2,500 or higher. Note that the base TWSC crash frequencies for these scenarios were based on the HSM method, which was expected to predict lower crashes for Nebraska scenarios. Therefore, actual crash-related benefit-cost ratios may be even higher than those predicted in the table. However, operationally, conversion with such high AADT on both major and minor approaches may not be feasible.

Overall, the analysis confirmed that RCUT conversion offers substantial safety and economic advantages at moderate- to high-volume rural expressway intersections, particularly where minor-road AADT exceeds 2,500 vehicles per day. At low-volume sites, however, the retrofit may not be economically justified unless construction costs are minimal. These findings support prioritizing RCUT implementation at high-exposure intersections where both major and minor approaches experience moderate to heavy traffic.

Table 8.11 shows the annual crash cost savings from implementing a multi-lane roundabout for different AADT combinations, along with the corresponding benefit-to-cost ratio for retrofit costs ranging from $250,000 to $7 million.

Table 8.11 Safety benefit-cost analysis of converting TWSC to multi-lane roundabouts

AADT (Major Approach)	AADT (Minor Approach)	TWSC Crash Frequency (per year)	RCUT Crash Frequency (per year)	Yearly Crash Costs Savings by roundabout	Benefit/Cost Ratio Retrofit Cost in millions (m)				
					$0.25 m	$1 m	$3 m	$5 m	$7 m
1,500	5,00	0.12	0.07	$29,894	1.3	0.3	0.1	0.1	0.0
1,500	2,500	1.42	0.80	$353,743	14.9	4.0	1.3	0.8	0.6
1,500	5,000	1.74	0.97	$433,459	18.2	4.9	1.6	1.0	0.7
1,500	7,500	2.13	1.19	$530,614	22.3	5.9	2.0	1.2	0.9
2,500	5,00	0.19	0.11	$47,332	2.0	0.5	0.2	0.1	0.1
2,500	2,500	1.54	0.86	$383,636	16.1	4.3	1.5	0.9	0.6
2,500	5,000	1.89	1.06	$470,826	19.8	5.3	1.8	1.1	0.8
2,500	7,500	2.31	1.29	$575,455	24.2	6.5	2.2	1.3	0.9
5,000	5,00	1.60	0.90	$398,583	16.7	4.5	1.5	0.9	0.7
5,000	2,500	1.89	1.06	$470,826	19.8	5.3	1.8	1.1	0.8
5,000	5,000	2.31	1.29	$575,455	24.2	6.5	2.2	1.3	0.9
5,000	7,500	2.84	1.59	$707,485	29.7	7.9	2.7	1.6	1.2
10,000	5,00	2.41	1.35	$600,366	25.2	6.7	2.3	1.4	1.0
10,000	2,500	2.84	1.59	$707,485	29.7	7.9	2.7	1.6	1.2
10,000	5,000	3.48	1.95	$866,919	36.4	9.7	3.3	2.0	1.4
10,000	7,500	4.28	2.40	$1,066,210	44.8	12.0	4.0	2.4	1.7
15,000	5,00	3.63	2.03	$904,286	38.0	10.1	3.4	2.1	1.5
15,000	2,500	4.28	2.40	$1,066,210	44.8	12.0	4.0	2.4	1.7
15,000	5,000	2.45	1.37	$610,331	25.6	6.8	2.3	1.4	1.0
15,000	7,500	2.94	1.65	$732,397	30.8	8.2	2.8	1.7	1.2
20,000	5,00	1.12	0.63	$279,008	11.7	3.1	1.1	0.6	0.5
20,000	2,500	2.30	1.29	$572,963	24.1	6.4	2.2	1.3	0.9
20,000	5,000	3.13	1.75	$779,728	32.8	8.7	3.0	1.8	1.3
20,000	7,500	3.75	2.10	$934,179	39.3	10.5	3.5	2.1	1.5
25,000	5,00	1.35	0.76	$336,305	14.1	3.8	1.3	0.8	0.5
25,000	2,500	2.77	1.55	$690,047	29.0	7.7	2.6	1.6	1.1
25,000	5,000	3.78	2.12	$941,653	39.6	10.6	3.6	2.2	1.5
25,000	7,500	4.54	2.54	$1,130,980	47.5	12.7	4.3	2.6	1.8

Note: *underlined values represent HSM-based crash frequency for TWSC intersection

Results from Table 8.11 indicate that both crash-reduction potential and economic return are strongly influenced by AADT levels on the major and minor approaches, as well as by varying retrofit costs.

With a major-road AADT of 2,500 and a minor-road AADT ranging from 500 to 7,500, the estimated annual crash-cost savings ranged from approximately $47,000 to $575,000. The corresponding benefit-cost ratios varied from 0.5 to 6.5 for a $1 million retrofit and from 0.1 to 1.3 for a $5 million retrofit. On the other hand, for a relatively moderate major-road AADT volume of 5,000 with minor-road AADT ranging from 500 to 7,500, the estimated annual crash-cost savings increased, ranging from approximately $399,000 to $707,000. The corresponding benefit-cost ratios varied from 4.5 to 7.9 for a $1 million retrofit and from 0.9 to 1.6 for a $5 million retrofit cost. Even when retrofit costs reached $7 million, the project maintained a favorable benefit-cost ratio above 1.0 for higher minor-road volumes. For balanced traffic conditions, such as when both the major and minor roads carry 5,000 AADT, the safety benefit-cost ratios were estimated at 24.2, 6.5, 2.2, 1.3, and 0.9 for retrofit costs of $0.25, $1, $3, $5, and $7 million, respectively. This analysis demonstrated that roundabouts are feasible from a safety standpoint for balanced traffic conditions. Similarly, operation-wise, roundabouts are known to perform efficiently under balanced traffic conditions, providing smooth flow with minimal delays.

When the major-road AADT is around 10,000 and the minor-road AADT ranges from 500 to 7,500, annual crash-cost savings ranged from $600,000 to more than $1 million. The corresponding benefit-cost ratios varied from 1.0 to 44.8, depending on the retrofit costs. Also, for a higher major-road AADT of 15,000, the benefit-cost ratios remained above 1 for all retrofit costs. This analysis demonstrated that, for major-road AADT between 10,000 and 15,000 and all minor-road AADT combinations, multi-lane roundabouts are economically feasible in terms of safety benefits. Furthermore, these AADT ranges were found to be operationally suitable for roundabout alternatives in Nebraska, based on the findings of this project.

At higher major-road volumes (20,000 to 25,000 AADT), annual crash-cost savings remain substantial, ranging from roughly $280,000 to $1.13 million. For these higher-volume intersections, benefit-cost ratios consistently exceed 1.0 for nearly all cost scenarios when the minor-road AADT is 2,500 or higher. While these traffic scenarios produced favorable results for economic safety and feasibility of roundabouts, precautions are needed to address the adverse impacts of traffic operations.

From the review of retrofit costs for multi-lane roundabouts and RCUTs, it was found that, in general, RCUTs have lower construction costs than multi-lane roundabouts. However, multi-lane roundabouts may be more suitable for a wider range of traffic conditions and have greater potential for crash reduction, according to the CMFs reviewed. Therefore, the benefit-cost analyses conducted on safety and feasibility in Tables 8.10 and 8.11provided valuable guidance for selecting the most appropriate intersection alternative under Nebraska traffic conditions.

8.5.2 Benefit-Cost Analysis of Interchange Safety

This section discussed the economic feasibility of converting DI/CDI to DDI for varying ramp and cross-street AADTs. The safety benefit-cost analysis was conducted using Equations (8.4) to (8.5).

Table 8.12 shows the year-by-year crash cost savings by DDI and the corresponding benefit-cost ratios for retrofit costs ranging from $1 million to $15 million.

Table 8.12 Safety benefit-cost analysis of converting DI/CDI to DDI configuration

AADT		Yearly Crash Costs Savings by DDI	Benefit/Cost Ratio				
			Retrofit Cost in millions (m)				
Ramp	Cross-street		$1 m	$3 m	$5 m	$10 m	$15 m
2,500	2,500	$415,245	4.5	1.6	0.9	0.5	0.3
	5,000	$572,524	6.2	2.1	1.3	0.7	0.4
	10,000	$791,263	8.6	3.0	1.8	0.9	0.6
	15,000	$886,412	9.6	3.3	2.0	1.0	0.7
	20,000	$974,612	10.6	3.7	2.2	1.1	0.7
	25,000	$1,057,117	11.5	4.0	2.4	1.2	0.8
	30,000	$1,137,391	12.3	4.3	2.6	1.3	0.9
5,000	2,500	$496,359	5.4	1.9	1.1	0.6	0.4
	5,000	$639,342	6.9	2.4	1.5	0.7	0.5
	10,000	$880,104	9.5	3.3	2.0	1.0	0.7
	15,000	$1,094,472	11.9	4.1	2.5	1.2	0.8
	20,000	$1,214,543	13.2	4.6	2.8	1.4	0.9
	25,000	$1,289,309	14.0	4.8	2.9	1.5	1.0
	30,000	$1,360,624	14.8	5.1	3.1	1.6	1.0
7,500	2,500	$564,436	6.1	2.1	1.3	0.6	0.4
	5,000	$703,804	7.6	2.6	1.6	0.8	0.5
	10,000	$930,237	10.1	3.5	2.1	1.1	0.7
	15,000	$1,130,430	12.3	4.2	2.6	1.3	0.9
	20,000	$1,316,121	14.3	4.9	3.0	1.5	1.0
	25,000	$1,491,262	16.2	5.6	3.4	1.7	1.1
	30,000	$1,558,780	16.9	5.8	3.5	1.8	1.2

Table 8.12 shows that for a ramp-AADT of 2,500, the yearly crash cost savings ranged from approximately $415,000 to $1,138,000 for cross-street AADT from 2,500 to 30,000. Considering retrofit costs ranging from $1 million to $5 million, the benefit-cost ratios ranged from 0.9 to 12.3. For retrofit costs of $10 million, only interchanges with cross-street AADT 15,000 or higher were deemed economically justified. With higher retrofit costs of $15 million, conversion is no longer feasible.

For moderate ramp-AADT of 5,000, the annual crash cost savings were more pronounced, ranging from approximately $496,000 to $1,136,000 for various cross-street AADTs. For retrofit costs of $5 million, the benefit-cost ratio ranged from 1.1 to 3.1 as cross-street AADT increased from 2,500 to 30,000. Therefore, the increase in cross-street volume helped justify the DDI choice, as it could potentially reduce more crashes. For higher retrofit costs of $10 million and $15 million, the benefit-cost ratios crossed 1.0 only when the cross-street AADT reached or exceeded 10,000 and 25,000, respectively.

A ramp AADT of 7,500 represented the most favorable scenario for justifying the DDI configuration, yielding the highest benefit-cost ratio among all scenarios analyzed. The estimated annual crash-cost savings ranged from $560,000 to $1,558,000. For retrofit costs up to $10 million, most traffic scenarios supported conversion to a DDI. However, when retrofit costs increased to $15 million, a cross-street AADT of approximately 20,000 was required for the DDI option to remain feasible.

8.6 Benefit-Cost Analysis of Grade Separation

Chapter 7 discussed the operational and safety impacts of converting an at-grade intersection (i.e., TWSC intersection) to a grade-separated diamond interchange. This section analyzed the economic feasibility of this conversion decision based on crash-costs and operational delay savings.

Table 8.13 shows the results of the benefit-cost analysis for varying AADTs of major and minor approaches with three retrofit costs of $10 million, $20 million, and $30 million.

Table 8.13 Operational and safety benefit-cost analysis of grade separations as alternative

AADT (major approach)	AADT (minor approach)	Benefit-Cost Ratio								
		Retrofit Cost in millions (m)								
		$10 m			$20 m			$30 m		
		Op.	Sa.	Cm.	Op.	Sa.	Cm.	Op.	Sa.	Cm.
1,500	5,00	-	-	-	-	-	-	-	-	-
1,500	2,500	-	-	-	-	-	-	-	-	-
1,500	5,000	-	-	-	-	-	-	-	-	-
1,500	7,500	-	-	-	-	-	-	-	-	-
2,500	5,00	-	-	-	-	-	-	-	-	-
2,500	2,500	-	-	-	-	-	-	-	-	-
2,500	5,000	-	-	-	-	-	-	-	-	-
2,500	7,500	-	-	-	-	-	-	-	-	-
5,000	5,00	-	-	-	-	-	-	-	-	-
5,000	2,500	-	-	-	-	-	-	-	-	-
5,000	5,000	-	-	-	-	-	-	-	-	-
5,000	7,500	-	-	-	-	-	-	-	-	-
10,000	5,00	-	-	-	-	-	-	-	-	-
10,000	2,500	-	-	-	-	-	-	-	-	-
10,000	5,000	-	-	-	-	-	-	-	-	-
10,000	7,500	4.2	-	4.2	2.1	-	2.1	1.4	-	1.4
15,000	5,00	-	-	-	-	-	-	-	-	-
15,000	2,500	-	-	-	-	-	-	-	-	-
15,000	5,000	3.7	-	3.7	1.8	-	1.9	1.2	-	1.2
15,000	7,500	7.4	-	7.4	3.7	-	3.7	2.5	-	2.5
20,000	5,00	-	-	-	-	-	-	-	-	-
20,000	2,500	1.6	-	1.7	-	-	-	-	-	-
20,000	5,000	4.1	-	4.1	2.0	-	2.0	1.3	-	1.4
20,000	7,500	13.0	-	13.0	6.5	-	6.5	4.3	-	4.3
25,000	5,00	-	-	-	-	-	-	-	-	-
25,000	2,500	4.5	-	4.5	2.3	-	2.3	1.5	-	1.5
25,000	5,000	9.1	-	9.1	4.5	-	4.6	3.0	-	3.0
25,000	7,500	22.3	-	22.4	11.2	-	11.2	7.4	-	7.5

Note: Op. (operation), Sa. (safety), Cm. (Operations and safety combined)
Empty cells (-) indicate benefit-cost ratio of less than 1.

Table 8.13 shows that, across the range of retrofit costs evaluated, the benefit-cost ratio based on crash cost savings alone never exceeded 1.0, indicating that no safety-related economic benefit was realized. A CMF of 0.92, as discussed in Chapter 7, was applied to estimate crash reduction benefits for conversion to a grade-separated facility. This translated into annual crash-cost savings ranging from approximately $5,400 to $205,600. However, compared with the substantial conversion costs associated with grade separation, these safety benefits were insufficient to yield a favorable benefit-cost ratio.

As a result, the analysis shifted focus to evaluating the operational feasibility of such conversions. Table 8.13 also presents the benefit-cost ratios derived from operational performance under varying traffic conditions. The results indicated that, from an operational standpoint, conversion to a grade-separated interchange becomes feasible under the following specific volume conditions: when the major-road AADT is 10,000 with a minor-road AADT of 7,500; when the major-road AADT is 15,000 with minor-road AADT of 5,000 or higher; and when the major-road AADT is 20,000 or 25,000 with minor-road AADT of 2,500 or more. Note that, for these volume scenarios, the at-grade intersection had LOS of F.

It is important to note that if a more favorable CMF, such as 0.58 from the HSM (shown in Table 7.2), had been used, the estimated crash-cost savings would have been higher, thereby improving the safety feasibility of the conversion. In addition, the at-grade intersection crash estimates derived from the negative binomial model without applying the volume threshold (included in Appendix D) would have resulted in higher crash frequencies, further increasing the safety benefits compared to those presented in Table 8.13. Nonetheless, a conservative approach was adopted by using a lower CMF (developed from a recent study by Srinivasan et al., 2025) and by relying on methodologically sound crash estimates to avoid overestimating the potential benefits.

Further analysis showed that when a CMF of 0.58 was applied, grade separation would be considered safety-feasible (excluding operational benefits) only if the existing at-grade intersection experienced at least 3.7, 7.4, and 11 crashes per year for retrofit costs of \$10 million, \$20 million, and \$30 million, respectively. When a less favorable CMF of 0.92 was used, the at-grade intersection would require a minimum of approximately 19.3 crashes per year for a $10 million retrofit to be justified from a safety perspective.

Chapter 9 Guidelines

This chapter discussed the final guidelines for intersections, interchanges, and grade separation choices for rural expressways in Nebraska.

9.1 Guidelines for Intersection

9.1.1 Intersection Guidelines Based on Operations

Using the logistic regression results from Table 8.8 and using a retrofit cost of $1 million, Figure 9.1 illustrates the predicted selection probabilities for alternative intersection types under varying traffic conditions using heat maps. The first, second, and third columns in the figure correspond to Model 1 (probability of selecting a roundabout over a TWSC), Model 2 (probability of selecting an RCUT over a TWSC), and Model 3 (probability of selecting a roundabout over an RCUT), respectively. These heat maps allow evaluation of operational feasibility by showing how combinations of major-road AADT, minor-road AADT, and left-turn percentage influence preferred intersection types. Scenarios where the predicted probability exceeds 0.5, indicated by the dashed line in each heat map, represent traffic conditions under which a conversion to the corresponding alternative is recommended. For example, when the dashed line in Model 1 (i.e., first column in the figure) is crossed (reddish to greenish area of plot), a roundabout becomes feasible compared to the TWSC intersection.

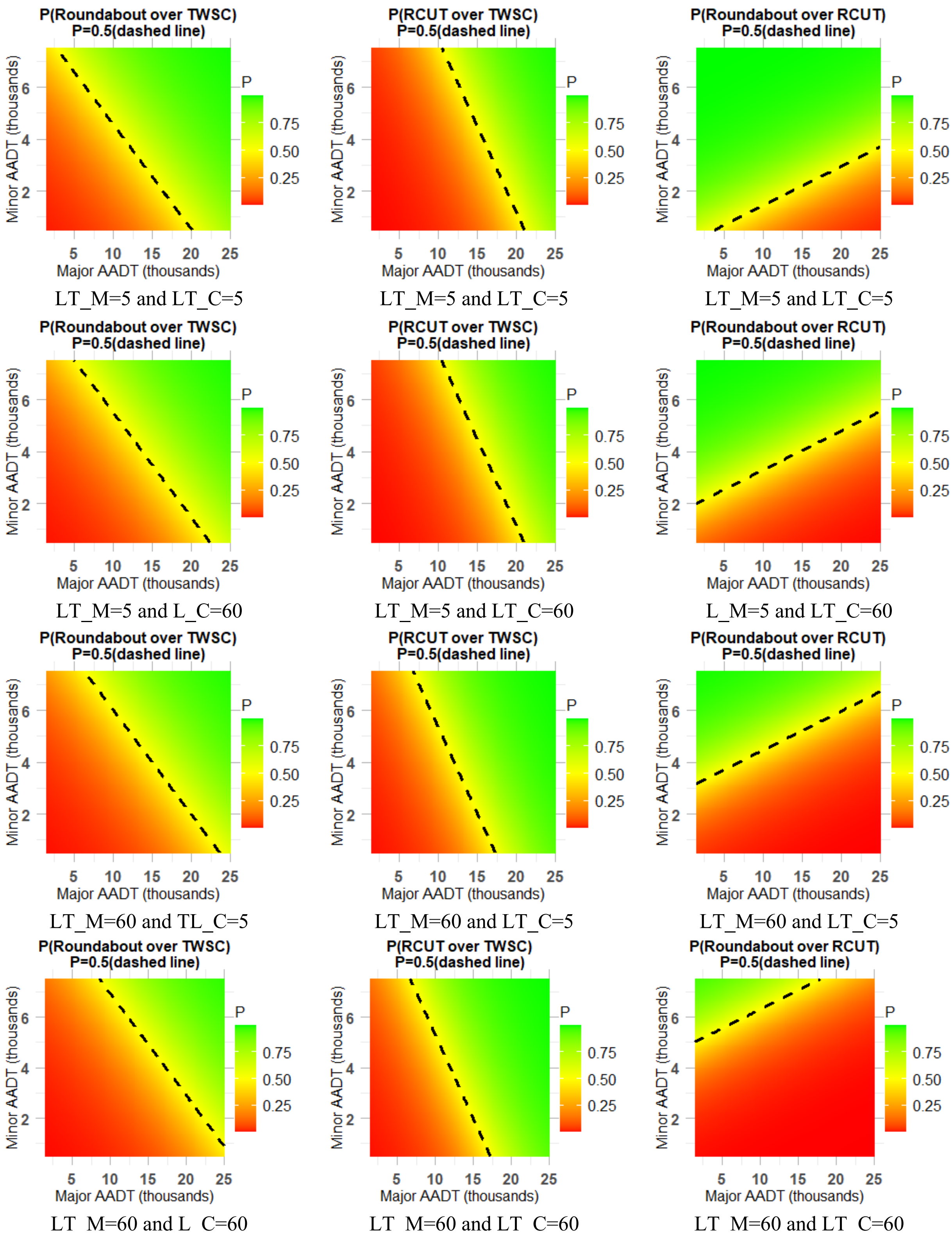


Figure 9.1 Unsignalized intersection choice thresholds based on $1 million conversion costs

Conversion guidelines from TWSC to roundabout

Retrofit cost of $1 million:

1. For low left-turning scenarios, the roundabout becomes viable if the minimum minor-AADT is 4,000; 2,500; and 500 for a major-AADT of 10,000; 15,000; and 20,000, respectively; beyond 20,000, a roundabout is favored irrespective of minor traffic volume.
2. For high left-turning scenarios, a roundabout becomes feasible if the minimum minor-AADT is 7,000; 5,000; 2,500; and 500 for a major-AADT of 10,000; 15,000; 20,000; and 25,000, respectively.

Retrofit cost of $5 million or higher:

3. For sites with regular turning characteristics (such as major road: L/T/R:20/70/10 and minor road L/T/R:40/40/20), for retrofit costs of $5 million or higher, the roundabout becomes feasible for a minor road AADT of 2,500; 5,000; and 7,500 when the major road AADT reaches or exceeds 20,000; 15,000; and 10,000 AADT.

Peak hour period recommendation:

4. While a roundabout may be economically feasible compared to a TWSC under these conditions, its peak hour operation may deteriorate when the major-road AADT reaches 15,000 with a minor-road AADT of 7,500 and high left-turn volumes, when the major-road AADT reaches 20,000 with high left-turn volumes, and when the major-road AADT reaches 25,000 with medium to high left-turn volumes.

Conversion guidelines from TWSC to RCUT

Retrofit cost of $1 million:

1. A TWSC remains feasible up to 10,000 major-AADT for low and 6,000 for high left-turning conditions.
2. An RCUT is preferred beyond 21,000 and 17,000 major-AADT for low and high left-turning conditions, regardless of minor volume.
3. For low left-turning scenarios, the RCUT is suitable if the minor-AADT is at least 7,500; 4,500; and 1,000 for major-AADT of 10,000; 15000; and 20,000, respectively.
4. For high left-turning scenarios, conversion to an RCUT is feasible for minimum minor-AADT of 7,000, 5,000, and 2,000 for major-AADT of 6,000, 10,000, and 15,000.

Retrofit cost of $5 million or higher:

5. For sites with regular turning characteristics (such as major road: L/T/R:20/70/10 and minor road L/T/R:40/40/20), and retrofit costs of $5 million or higher, RCUT becomes feasible for minor road AADT of 2,500; 5000; and 7,500 when the major road AADT reaches or exceeds 20,000; 15,000; and 10,000 AADT.

Peak hour period recommendation:

6. While an RCUT may be economically feasible compared to a TWSC under these conditions, its peak hour operation may deteriorate when the major-road AADT reaches 15,000 with a minor-road AADT of 7,500 and high through and left-turn volume combinations, when the major-road AADT reaches 20,000 with minor-road AADT of 5,000 or over, and when the major-road AADT reaches 25,000 with minor road AADT of 2,500 or higher.

Conversion guidelines from RCUT to Roundabout (retrofit costs of $1 million)

To the best of the authors' knowledge, RCUT installations have not been converted into roundabouts in practice. However, a limited number of studies have examined the feasibility of such conversions. For locations with potential future changes in land use, a roundabout may be a viable replacement for an existing RCUT when signalization of the RCUT is not pursued. The following guidelines consider scenarios in which conversion from an RCUT to a roundabout.

1. For low left-turning conditions, conversion to a roundabout is viable if minor-AADT is at least 500; 1,000; 2,000; 3,000; and 4,000 for major-AADT of 5,000; 10,000; 15,000; 20,000; and 25,000.
2. For high left-turning sites, at least 5,000 minor-AADT is required for roundabout consideration, but it becomes infeasible beyond 17,5000 major-AADT.
3. For major-AADT of 5,000; 10,000; 15,000; and 17,500; minor-AADT should be at least 5,500; 6,500; 7,000; and 7,500, respectively, for roundabout consideration.

9.1.2 Intersection Guidelines Based on Safety

Conversion guidelines from TWSC to roundabout

1. For a lower retrofit cost of $1 million, roundabout conversion can be considered feasible when the major-road AADT reaches 2,500 and minor-road AADT exceeds 500.
2. For more typical retrofit costs of $5 million or higher, a roundabout may not be feasible unless the major-road AADT exceeds 5,000.
3. For locations with a major-road AADT of 10,000 or higher, roundabout conversion should be considered feasible for retrofit costs up to $7 million.

4. For a higher retrofit cost of around $10 million, sites with a major-road AADT of 10,000 accompanied by high minor-road traffic, or locations with a major-road AADT of 15,000 or higher, should be considered candidates for conversion to a roundabout.

<u>Conversion guidelines from TWSC to RCUT</u>

1. For a lower retrofit cost of $1 million, RCUT conversion can be considered feasible when the major-road AADT reaches 5,000 with the minor-road AADT of 500 or higher.
2. For more typical costs of $3 million, RCUT conversion should be feasible for major-road AADT of 5,000 with minor-road AADT of 500. Beyond this threshold, RCUT will remain feasible for the $3 million retrofit cost.
3. For higher costs of $5 million or greater, RCUT conversion may not be feasible when the major-road AADT is 5,000 or less.
4. For locations with a major-road AADT of 10,000, RCUT conversion should be considered feasible for retrofit costs up to $5 million. When retrofit costs approach $7 million, a minor-road AADT of at least 5,000 should be expected for feasibility.
5. For major-road AADT values of 15,000 or higher, RCUT conversion should be considered feasible for retrofit costs up to $7 million when the minor-road AADT is at least 500.

It is important to note that retrofit decisions should be based on both operational and safety implications when information for both is available. Because crash costs, particularly those associated with fatal and injury outcomes, are substantially higher, a safety-based benefit-cost analysis may indicate that an alternative is economically feasible even when operational performance does not support the same choice. Hence, careful judgment should be practiced when choosing such alternatives. At the same time, an alternative that presents safety concerns should not be selected even if operational efficiency appears acceptable.

9.2 Guidelines for Interchange

9.2.1 Interchange Guidelines Based on Operations

Using the logistic regression results from Table 8.9and assuming various retrofit costs, Figures 9.2 and 9.3 illustrate the predicted selection probabilities for alternative interchange types under varying traffic conditions using heat maps. The first, second, and third columns in the figure correspond to Model 4 (probability of selecting a DIsig over a DIstop), Model 5 (probability of selecting a DDI over a DDIstop), and Model 6 (probability of selecting a DDI over DIsig), accordingly, for retrofit costs of \$5 million, \$5 million, and \$3 million (m), respectively. These heat maps allow evaluation of operational feasibility by showing how combinations of major-road AADT, minor-road AADT, and left-turn percentage influence preferred interchange types. Scenarios where the predicted probability exceeds 0.5, indicated by the dashed line in each heat map, represent traffic conditions under which a conversion to the corresponding alternative is recommended. Figure 9.2 emphasizes conversions based on AADT of both approaches and Figure 9.3 focuses on the cross-street's AADT and left-turn proportions.

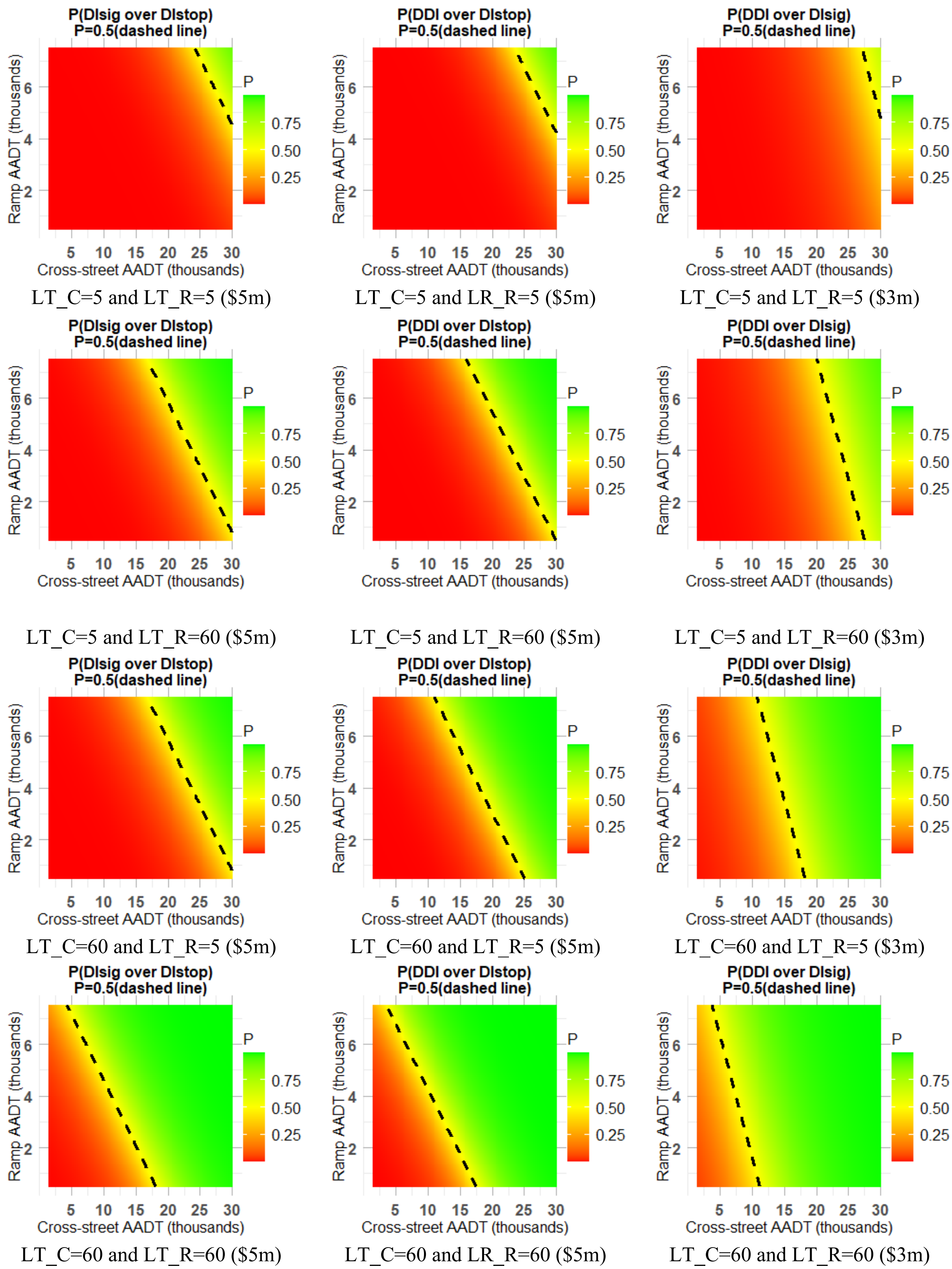


Figure 9.2 Interchange choice thresholds for cross-street and ramp AADT based on moderate conversion costs

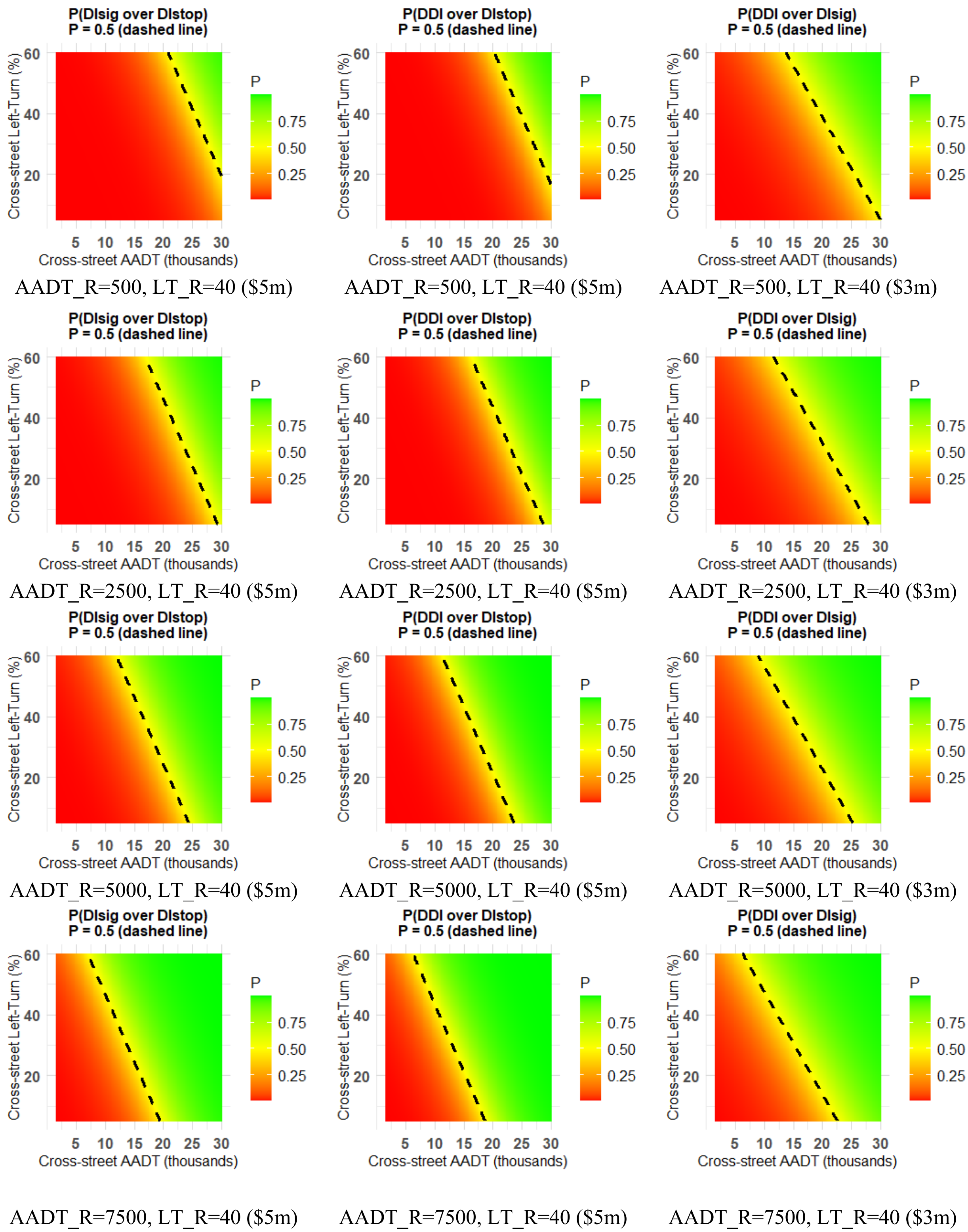


Figure 9.3 Intersection choice thresholds for cross-street left-turn and AADT based on moderate conversion costs

While Figures 9.2 and 9.3 can be used for scenario-specific guidance, the following presents overall guidelines. It was found that the conditions for converting from DIstop to DIsig or DDI were quite similar; therefore, their conversion guidelines are presented together.

<u>Conversion guidelines from DIstop to DIsig or DDI</u>

Retrofit cost $5 million:

1. For low left-turn scenarios, a DIstop is feasible for up to a 22,500 cross-street AADT; for high left-turn scenarios, DIstop may not be feasible after the cross-street reaches 5,000 AADT.
2. For low left-turn scenarios, beyond 20,000 cross-street AADT, DIsig/DDI can be feasible options provided that at least 4,500 to 7,500 ramp AADT exists.
3. For high left-turn conditions, DIsig/DDI becomes feasible if the ramp-AADT is at least 7,000; 4,500; and 2,500 for cross-street-AADT of 5,000; 10,000; and 15,000; beyond 18,000 cross-street-AADT, DIsig/DDI is the preferred option regardless of ramp volume.

Retrofit costs $10 million or higher:

4. DDI/DIsig remains feasible for sites with a ramp AADT of 500 to 2,500 and cross-street AADT of at least 25,000. For higher ramp volumes of 5,000 and 7,500 AADT, cross-street traffic should be at least 20,000 and 15,000, respectively, for DDI to be feasible under high retrofit costs.

Peak hour period recommendation:

5. While a DIsig may be economically feasible compared to a DIstop under these conditions, its peak-hour operation may deteriorate in cross-street AADT of 25,000 or higher with heavy left-turn scenarios. For DDI as the alternative, such operational issues may arise in the high left-turn scenario for cross-street AADT of 30,000 or higher.

Conversion from DIsig to DDI

Retrofit cost $3 million:

1. For low left-turn scenarios for both cross-street and ramp approaches, converting to DDI is not an operationally feasible option compared to DIsig.
2. For low left-turn cross-street and high left-turn ramp conditions, DDI is viable beyond 27,500 cross-street-AADT. On the other hand, for high left-turn cross-street and low left-turn ramp scenarios, DDI is economically feasible for cross-streets with over 18,000 AADT.
3. For high-turn scenarios for both approaches, DDI may be feasible with cross-street AADT as low as 2,500 if there is a high ramp-AADT condition. For cross-street AADT of 5,000; 7,500; and 10,000, a minimum ramp traffic of 6,500; 4,500; and 1,500, respectively, is required for DDI to be viable compared to DIsig. Beyond 12,500 cross-street AADT, a DDI is viable regardless of ramp traffic volume.

Retrofit costs $10 million or higher:

4. With higher retrofit costs, converting DDI from DIsig is no longer economically feasible at low to medium left-turn volumes. However, for sites with high left-turn traffic, DDI becomes feasible when cross-street AADT reaches 20,000 for high ramp volume and 25,000 for low to moderate ramp traffic.

Peak hour period recommendation:

5. DDI may face operational issues for high left-turn scenarios for cross-street AADT of 30,000 or higher.

9.2.2 Interchange Guidelines Based on Safety

The following guidelines are for converting DI/CDI (i.e., DIstop/DIsig) to DDI.

Lower Ramp AADT of 2,500

1. For retrofit costs up to $5 million, DDI conversion can be considered feasible across a broad range of cross-street traffic volumes.
2. When retrofit costs reach $10 million, only sites with cross-street AADT of 15,000 or higher should be considered economically justified.
3. At retrofit costs of $15 million, DDI conversion is not considered feasible for this ramp volume level.

Moderate Ramp AADT of 5,000

1. For retrofit costs of around $5 million, DDI conversion should be considered feasible for most cross-street AADT conditions.
2. For retrofit costs of around $10 million, feasibility should be expected only when the cross-street AADT reaches or exceeds 10,000.
3. For retrofit costs around $15 million, feasibility should be considered only when cross-street AADT approaches or exceeds 25,000.

High Ramp AADT of 7,500

1. This high ramp volume provided the strongest economic justification for a DDI due to substantial safety benefits.
2. For retrofit costs up to $10 million, most cross-street AADT scenarios should be considered feasible for DDI conversion.
3. For retrofit costs around $15 million, conversion should be considered feasible only when cross-street AADT is approximately 20,000 or higher.

9.3 Guidelines for Signalization

TWSC intersections may need to be retrofitted to signalized intersections for certain traffic conditions. On the other hand, often signalized intersections are converted into roundabouts. This section presents guidelines for signalization suitable for rural expressways.

1. Performance due to signalization is highly dependent on lane configuration, signal timing and phasing, and local traffic characteristics. Site-specific studies are more suitable for performance evaluation than generalized guidelines.
2. Signalization should be avoided at low-volume rural expressway intersections**,** since multiple signal phases interrupt major-road flow and may not provide operational benefits. At such sites, TWSC intersections generally maintain acceptable LOS and are typically more suitable.
3. When minor-road AADT exceeds 5,000, signalized intersections without minor-road left-turn bays tend to experience substantial delays, often producing intersection LOS of D or F during peak hours. Under these conditions, TWSC and roundabout alternatives generally provide better operational performance.
   Consequently, for minor-road AADT greater than 5,000, a left-turn bay on the minor approach legs should be provided before signalization is considered, as it significantly improves operational performance at the signalized intersection.

4. When minor-road AADT is below 5,000, TWSC control typically maintains LOS A to B for major-road AADT up to 10,000 during peak periods. Signalization may begin to offer operational advantages only when major-road AADT exceeds 10,000 and minor-road AADT remains moderate. However, roundabouts and RCUTs still remain more suitable

candidates than signalized intersections for sites with major-road AADT of 10,000 or higher.

5. Signalization may be more appropriate once major-road AADT reaches 15,000 or higher, especially when roundabout performance begins to deteriorate due to high directional flow or imbalanced volumes between approaches, and RCUTs may suffer from high through and left-turn volumes.
6. For major-road AADT values around 20,000, a left-turn bay on the minor approach should be considered essential for feasible signal operation, even when minor-road AADT is low to moderate.
7. Roundabouts generally outperform signalized intersections when major and minor traffic volumes are balanced, providing LOS A under conditions such as 2,500 per approach or 5,000 per approach. Under these traffic patterns, signalized intersections typically provide LOS C to D, making roundabouts a more suitable option.
8. When AADT becomes highly unbalanced, with major-road traffic far exceeding minor-road traffic, roundabout performance declines due to increased delays on minor approaches. Under such conditions, signalization may provide more consistent LOS.
9. Safety benefits of signalization are contested. While earlier studies suggested potential improvements to signalization, recent Midwest-based studies suggested that signalization may contribute to more crashes, specifically at high-speed intersections.
10. Along rural expressways in Nebraska, roadway segments rarely exceed 15,000 AADT, making unsignalized control the more practical option in most locations. Signalization should therefore be reserved for site-specific cases where major-road demand is sufficiently high and local operational conditions justify the need. Given both operational

and safety implications, it is recommended that, before installing signals at high-speed intersections, alternative treatments such as roundabouts and RCUTs be carefully evaluated. Even grade separation may be an acceptable option, although it is often cost-prohibitive in many situations.

9.4 Guidelines for Grade Separation

Guidelines Based on Operational Studies:

1. Grade separation should be operationally feasible, considering the retrofit costs when the at-grade intersection, such as two-way stop-controlled intersections, operates under the following traffic demand conditions:
   - Major AADT of 10,000 with minor AADT of 7,500,
   - Major AADT of 15,000 with minor AADT of 5,000 or higher,
   - Major AADT of 20,000 or 25,000 with minor AADT of 2,500 or higher,
2. However, other at-grade alternatives such as roundabouts and RCUTs should be considered operationally adequate for traffic scenarios up to a major-road AADT of approximately 15,000. Grade separation options should be evaluated when the major-road AADT reaches or exceeds 20,000 and minor-road traffic demand is moderate to high. While signalized intersections may operate effectively at around 20,000 major-road AADT, these conditions may be associated with an increased likelihood of crashes; therefore, grade separation should be considered for such traffic demands.

Guidelines Based on Safety Studies:

1. On rural expressways in Nebraska, grade separation should be considered only as a last resort when other design options are no longer adequate.

2. While grade separation should be recognized for its ability to enhance safety performance, agencies should also account for the substantial construction and retrofit costs associated with such conversions.
3. Safety benefits alone may not be sufficient to justify grade separation, as they may not outweigh the high costs without support from operational benefits.
4. Using a favorable CMF such as 0.58, an at-grade facility with crash frequencies of approximately 3.7, 7.4, and 11.0 crashes per year would justify grade separation based on safety benefits alone for retrofit costs of about $10 million, $20 million, and $30 million, respectively.

   However, when applying a more conservative CMF such as 0.92, an at-grade site would require a much higher crash frequency, around 20 crashes per year, for grade separation to be justified at the lower retrofit cost tier.

Chapter 10 Summary, Conclusions, and Future Research

The Traffic Division of NDOT routinely faces decisions regarding whether an existing TWSC intersection on a rural expressway should be converted to another facility type, such as an RCUT, roundabout, signalized intersection, or interchange. As Nebraska continues toward completing its expressway system, these decisions will remain essential. The existing NDOT guidance for such selections was based on research from the 1990s and no longer reflects current traffic conditions, design practices, or safety performance. This project updated and expanded the guidance using recent data, contemporary analytical methods, and Nebraska-specific traffic and crash information.

Operational traffic data were collected from TWSC, RCUT, roundabout, and signalized intersections at rural locations across Nebraska, as well as from diamond interchanges with stop-controlled and signalized ramps and diverging diamond interchanges. These sites were modeled and evaluated using microsimulation models calibrated to Nebraska driving behavior. The study produced a comprehensive modeling effort that examined more than 11,500 traffic scenarios, incorporating a wide range of major-road, minor-road, and ramp volumes, turning movement combinations, and time-of-day conditions. The resulting operational outcomes were used to develop statistical models that allow quick performance estimation. Additional predictive models were constructed to identify the most suitable intersection or interchange type under varying traffic conditions.

Crash data for TWSC intersections along rural expressways from 2020 to 2024 were analyzed using a negative binomial model to estimate expected crash frequency under different traffic scenarios. Crash performance of alternative facilities such as roundabouts, RCUTs, and signalized intersections was assessed using appropriate CMFs. For grade-separated facilities, the

FHWA “Interchange Comparison Safety Tool” was applied to estimate crash frequencies for diamond interchanges and diverging diamond interchanges under Nebraska-representative traffic conditions.

The research also compiled construction and retrofit cost information for various intersection and interchange types from both national sources and recent Nebraska projects. NDOT supplied Nebraska-based crash-cost values for use in economic evaluation. Using these inputs, a comprehensive benefit-cost analysis was performed to assess the feasibility of converting TWSC intersections to alternative at-grade or grade-separated facilities.

The final guidelines for selecting intersection and interchange types were developed by integrating benefit-cost results with operational findings, safety performance, engineering judgment, and current practices. These updated guidelines provide NDOT with a modern, data-driven framework to support consistent, informed decisions on intersection upgrades, interchange selection, and grade-separation evaluations along Nebraska’s rural expressways.

While this project applied a comprehensive approach, several limitations should be acknowledged. Although a large number of traffic scenarios were evaluated, additional scenarios remain possible, including higher ramp volumes, more diverse turning-movement combinations, and AADT conditions with smaller differences between major and minor approaches. As more RCUT and DDI installations take place in Nebraska, Nebraska-specific safety performance functions can be developed to improve the accuracy of crash predictions. Furthermore, the analysis focused primarily on typical rural expressway intersections, assuming a two-lane major approach and a single-lane minor approach (for each direction); different lane configurations may yield different operational or safety outcomes. Similarly, the interchange analysis assumed a single ramp and a two-lane cross street, even though variations of these layouts exist in practice.

However, interpolation methods, such as the ratio of the number of lanes and AADT, may be used to infer outcomes for scenarios with alternative lane configurations not covered in this project.

Although this project focused on traditional at-grade and grade-separated facilities, several unconventional intersection and interchange forms were included, including RCUT and DDI installations. Future studies should build on this work by evaluating the feasibility and performance of additional unconventional designs already in use in Nebraska, as well as those not yet implemented in the state but applied in other regions. Such research would further strengthen NDOT’s ability to make informed decisions regarding intersection and interchange selection across a broader range of geometric and operational conditions.

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Appendix A Manual Traffic Volume Counts

This section presents the manual traffic volume count data collected at the various intersections and interchanges included in this project. The locations of data collection devices, their corresponding device names, and the traffic movements captured are illustrated in the accompanying figures, while detailed vehicle movement counts are provided in the subsequent tables.

The following abbreviations are used.

WBL: Westbound Left-Turn Movement

WBT: Westbound Through Movement

WBR: Westbound Right-Turn Movement

EBL: Eastbound Left-Turn Movement

EBT: Eastbound Through Movement

EBR: Eastbound Right-Turn Movement

NBL: Northbound Left-Turn Movement

NBT: Northbound Through Movement

NBR: Northbound Right-Turn Movement

SBL: Southbound Left-Turn Movement

SBT: Southbound Through Movement

SBR: Southbound Right-Turn Movement

U: U-Turn Movement

PC: Passenger Car

SUT/T: Single Unit Truck

TT: Tractor-Trailer Truck

TWSC: Two-Way Stop-Controlled Intersection

OWSC: One-way Stop-Controlled Intersection

SI: Signalized Intersection

RCUT: Restricted Crossing U-Turn Intersection

DIstop: Diamond Interchange with Stop-Controlled Ramp

DIsig: Diamond Interchange with Signal-Controlled Ramp

A.1 Intersection Volumes

A.1.1 TWSC at Hebron

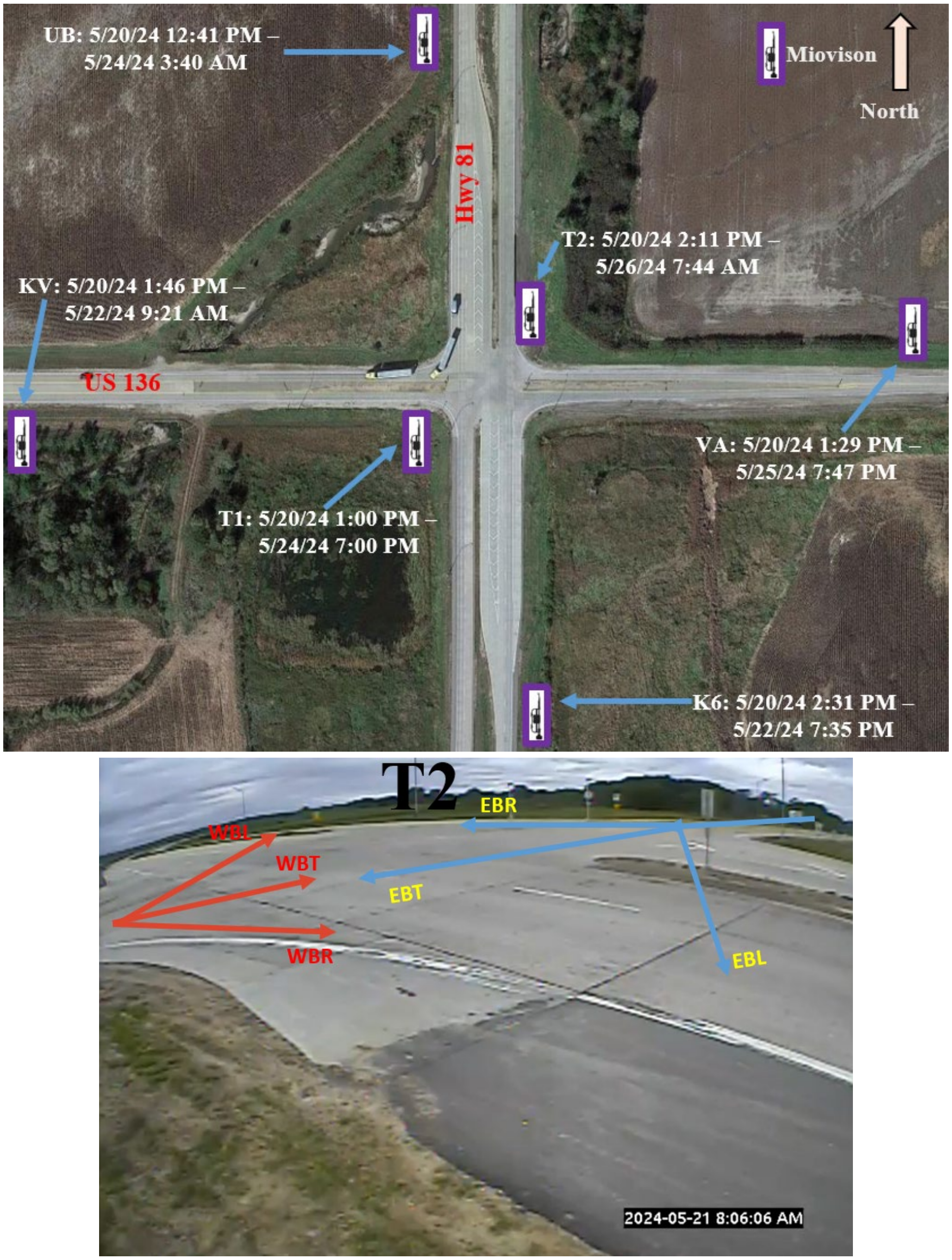


Figure A.1 Data collection device locations and corresponding traffic movements at the Hebron site

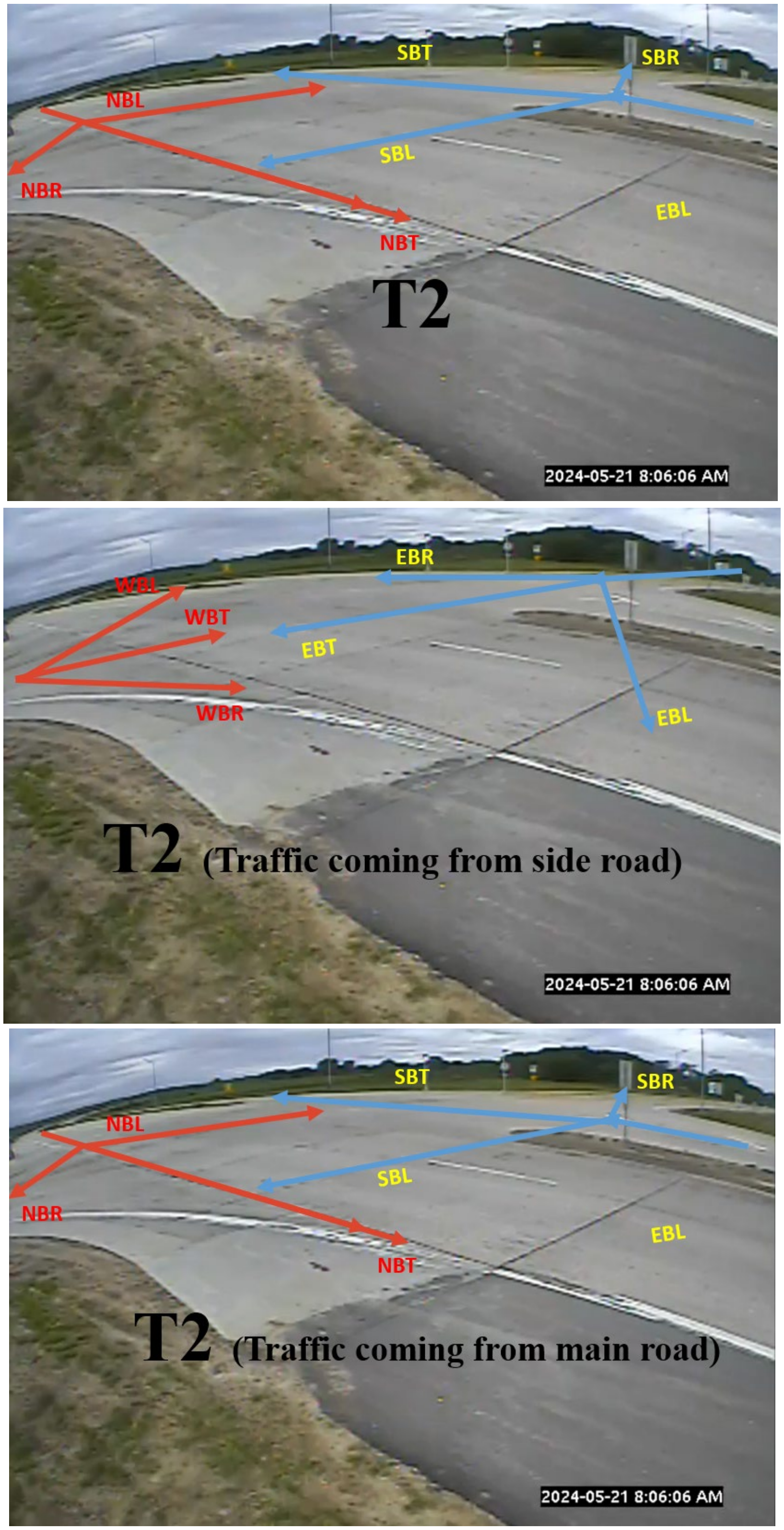


Figure A.1 cont. Data collection device locations and corresponding traffic movements at the Hebron site

Table A.1 Traffic volume counts at the Hebron site

Date: 5/21/2024

Recording Device	Lane Observed	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	WBL	8:00 am to 8:15 am	1	1	0	2
	WBT	8:00 am to 8:15 am	9	0	0	9
	WBR	8:00 am to 8:15 am	7	0	0	7
	EBR	8:00 am to 8:15 am	0	0	0	0
	EBT	8:00 am to 8:15 am	10	0	0	10
	EBL	8:00 am to 8:15 am	3	1	0	4
	NBR	8:00 am to 8:15 am	3	0	0	3
	NBT	8:00 am to 8:15 am	15	7	1	23
	NBL	8:00 am to 8:15 am	1	0	0	1
	SBL	8:00 am to 8:15 am	2	0	1	3
	SBT	8:00 am to 8:15 am	16	7	0	23
	SBR	8:00 am to 8:15 am	1	0	0	1
	WBL	8:15 am to 8:30 am	3	0	0	3
	WBT	8:15 am to 8:30 am	7	1	0	8
	WBR	8:15 am to 8:30 am	1	0	0	1
	EBR	8:15 am to 8:30 am	0	0	0	0
	EBT	8:15 am to 8:30 am	6	1	0	7
	EBL	8:15 am to 8:30 am	3	0	0	3
	NBR	8:15 am to 8:30 am	0	0	0	0
	NBT	8:15 am to 8:30 am	5	2	0	7
	NBL	8:15 am to 8:30 am	0	0	0	0
	SBL	8:15 am to 8:30 am	12	0	0	12
	SBT	8:15 am to 8:30 am	14	7	0	21
	SBR	8:15 am to 8:30 am	2	0	0	2
	WBL	8:30 am to 8:45 am	1	1	0	2
	WBT	8:30 am to 8:45 am	3	1	1	5
	WBR	8:30 am to 8:45 am	6	0	0	6
	EBR	8:30 am to 8:45 am	0	0	0	0
	EBT	8:30 am to 8:45 am	4	0	0	4
	EBL	8:30 am to 8:45 am	1	1	0	2
	NBR	8:30 am to 8:45 am	2	0	0	2
	NBT	8:30 am to 8:45 am	14	8	1	23
	NBL	8:30 am to 8:45 am	0	0	0	0
	SBL	8:30 am to 8:45 am	0	5	0	5
	SBT	8:30 am to 8:45 am	5	13	0	18
	SBR	8:30 am to 8:45 am	13	0	0	13
	WBL	8:45 am to 9:00 am	1	0	0	1
	WBT	8:45 am to 9:00 am	11	0	0	11
	WBR	8:45 am to 9:00 am	3	1	0	4
	EBR	8:45 am to 9:00 am	0	0	0	0
	EBT	8:45 am to 9:00 am	6	1	0	7
	EBL	8:45 am to 9:00 am	2	0	0	2
	NBR	8:45 am to 9:00 am	3	0	0	3
	NBT	8:45 am to 9:00 am	19	8	0	27
	NBL	8:45 am to 9:00 am	0	0	0	0
	SBL	8:45 am to 9:00 am	3	2	0	5
	SBT	8:45 am to 9:00 am	17	7	0	24
	SBR	8:45 am to 9:00 am	1	0	0	1

Recording Device	Lane Observed	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	WBL	9:00 am to 9:15 am	0	0	0	0
	WBT	9:00 am to 9:15 am	9	0	0	9
	WBR	9:00 am to 9:15 am	3	1	0	4
	EBR	9:00 am to 9:15 am	0	0	0	0
	EBT	9:00 am to 9:15 am	5	1	0	6
	EBL	9:00 am to 9:15 am	3	0	0	3
	NBR	9:00 am to 9:15 am	1	0	0	1
	NBT	9:00 am to 9:15 am	17	7	1	25
	NBL	9:00 am to 9:15 am	0	0	0	0
	SBL	9:00 am to 9:15 am	9	1	0	10
	SBT	9:00 am to 9:15 am	12	3	0	15
	SBR	9:00 am to 9:15 am	1	0	0	1
	WBL	9:15 am to 9:30 am	1	0	0	1
	WBT	9:15 am to 9:30 am	6	0	0	6
	WBR	9:15 am to 9:30 am	1	0	1	2
	EBR	9:15 am to 9:30 am	0	1	0	1
	EBT	9:15 am to 9:30 am	4	1	0	5
	EBL	9:15 am to 9:30 am	6	0	0	6
	NBR	9:15 am to 9:30 am	2	0	0	2
	NBT	9:15 am to 9:30 am	16	9	0	25
	NBL	9:15 am to 9:30 am	0	0	0	0
	SBL	9:15 am to 9:30 am	9	0	0	9
	SBT	9:15 am to 9:30 am	10	5	0	15
	SBR	9:15 am to 9:30 am	3	1	1	5
	WBL	9:30 am to 9:45 am	1	1	0	2
	WBT	9:30 am to 9:45 am	6	1	0	7
	WBR	9:30 am to 9:45 am	2	2	1	5
	EBR	9:30 am to 9:45 am	2	0	0	2
	EBT	9:30 am to 9:45 am	5	0	1	6
	EBL	9:30 am to 9:45 am	2	0	0	2
	NBR	9:30 am to 9:45 am	2	1	0	3
	NBT	9:30 am to 9:45 am	18	3	0	21
	NBL	9:30 am to 9:45 am	0	0	0	0
	SBL	9:30 am to 9:45 am	3	1	0	4
	SBT	9:30 am to 9:45 am	22	9	0	31
	SBR	9:30 am to 9:45 am	3	0	0	3
	WBL	9:45 am to 10:00 am	0	1	0	1
	WBT	9:45 am to 10:00 am	7	2	0	9
	WBR	9:45 am to 10:00 am	6	3	0	9
	EBR	9:45 am to 10:00 am	0	0	0	0
	EBT	9:45 am to 10:00 am	6	1	0	7
	EBL	9:45 am to 10:00 am	0	0	0	0
	NBR	9:45 am to 10:00 am	2	0	1	3
	NBT	9:45 am to 10:00 am	18	14	0	32
	NBL	9:45 am to 10:00 am	4	0	0	4
	SBL	9:45 am to 10:00 am	5	36	0	41
	SBT	9:45 am to 10:00 am	19	11	1	31
	SBR	9:45 am to 10:00 am	1	0	0	1

Recording Device	Lane Observed	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	WBL	4:00 pm to 4:15 pm	4	0	0	4
	WBT	4:00 pm to 4:15 pm	5	0	0	5
	WBR	4:00 pm to 4:15 pm	5	1	0	6
	EBR	4:00 pm to 4:15 pm	1	0	0	1
	EBT	4:00 pm to 4:15 pm	9	0	0	9
	EBL	4:00 pm to 4:15 pm	1	1	0	2
	NBR	4:00 pm to 4:15 pm	1	0	0	1
	NBT	4:00 pm to 4:15 pm	13	9	1	23
	NBL	4:00 pm to 4:15 pm	0	0	0	0
	SBL	4:00 pm to 4:15 pm	7	0	0	7
	SBT	4:00 pm to 4:15 pm	17	8	0	25
	SBR	4:00 pm to 4:15 pm	3	0	0	3
	WBL	4:15 pm to 4:30 pm	5	0	0	5
	WBT	4:15 pm to 4:30 pm	10	1	0	11
	WBR	4:15 pm to 4:30 pm	9	0	0	9
	EBR	4:15 pm to 4:30 pm	0	0	0	0
	EBT	4:15 pm to 4:30 pm	9	1	0	10
	EBL	4:15 pm to 4:30 pm	1	0	0	1
	NBR	4:15 pm to 4:30 pm	2	0	0	2
	NBT	4:15 pm to 4:30 pm	16	6	0	22
	NBL	4:15 pm to 4:30 pm	0	0	0	0
	SBL	4:15 pm to 4:30 pm	4	2	0	6
	SBT	4:15 pm to 4:30 pm	13	12	0	25
	SBR	4:15 pm to 4:30 pm	1	0	0	1
	WBL	4:30 pm to 4:45 pm	3	1	0	4
	WBT	4:30 pm to 4:45 pm	6	0	0	6
	WBR	4:30 pm to 4:45 pm	4	0	0	4
	EBR	4:30 pm to 4:45 pm	2	1	0	3
	EBT	4:30 pm to 4:45 pm	9	0	0	9
	EBL	4:30 pm to 4:45 pm	6	0	0	6
	NBR	4:30 pm to 4:45 pm	2	0	0	2
	NBT	4:30 pm to 4:45 pm	25	13	1	39
	NBL	4:30 pm to 4:45 pm	0	0	0	0
	SBL	4:30 pm to 4:45 pm	10	1	0	11
	SBT	4:30 pm to 4:45 pm	25	9	0	34
	SBR	4:30 pm to 4:45 pm	1	0	0	1
	WBL	4:45 pm to 5:00 pm	1	0	0	1
	WBT	4:45 pm to 5:00 pm	7	0	0	7
	WBR	4:45 pm to 5:00 pm	4	1	0	5
	EBR	4:45 pm to 5:00 pm	0	0	0	0
	EBT	4:45 pm to 5:00 pm	15	1	0	16
	EBL	4:45 pm to 5:00 pm	4	0	0	4
	NBR	4:45 pm to 5:00 pm	3	0	0	3
	NBT	4:45 pm to 5:00 pm	23	6	0	29
	NBL	4:45 pm to 5:00 pm	0	0	0	0
	SBL	4:45 pm to 5:00 pm	4	0	0	4
	SBT	4:45 pm to 5:00 pm	15	10	0	25
	SBR	4:45 pm to 5:00 pm	2	1	0	3

Recording Device	Lane Observed	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	WBL	5:00 pm to 5:15 pm	1	0	0	1
	WBT	5:00 pm to 5:15 pm	8	0	0	8
	WBR	5:00 pm to 5:15 pm	2	0	0	2
	EBR	5:00 pm to 5:15 pm	0	0	0	0
	EBT	5:00 pm to 5:15 pm	15	0	0	15
	EBL	5:00 pm to 5:15 pm	4	0	0	4
	NBR	5:00 pm to 5:15 pm	1	1	0	2
	NBT	5:00 pm to 5:15 pm	15	14	0	29
	NBL	5:00 pm to 5:15 pm	0	0	0	0
	SBL	5:00 pm to 5:15 pm	17	0	0	17
	SBT	5:00 pm to 5:15 pm	18	11	0	29
	SBR	5:00 pm to 5:15 pm	0	1	0	1
	WBL	5:15 pm tp 5:30 pm	2	0	0	2
	WBT	5:15 pm tp 5:30 pm	9	0	0	9
	WBR	5:15 pm tp 5:30 pm	4	0	0	4
	EBR	5:15 pm tp 5:30 pm	1	0	0	1
	EBT	5:15 pm tp 5:30 pm	11	0	0	11
	EBL	5:15 pm tp 5:30 pm	4	0	0	4
	NBR	5:15 pm tp 5:30 pm	1	0	0	1
	NBT	5:15 pm tp 5:30 pm	17	10	0	27
	NBL	5:15 pm tp 5:30 pm	0	0	0	0
	SBL	5:15 pm tp 5:30 pm	5	0	0	5
	SBT	5:15 pm tp 5:30 pm	17	18	0	35
	SBR	5:15 pm tp 5:30 pm	8	0	0	8
	WBL	5:30 pm to 5:45 pm	0	0	1	1
	WBT	5:30 pm to 5:45 pm	13	0	0	13
	WBR	5:30 pm to 5:45 pm	8	0	0	8
	EBR	5:30 pm to 5:45 pm	0	0	0	0
	EBT	5:30 pm to 5:45 pm	8	0	0	8
	EBL	5:30 pm to 5:45 pm	2	0	0	2
	NBR	5:30 pm to 5:45 pm	0	2	0	2
	NBT	5:30 pm to 5:45 pm	21	18	2	41
	NBL	5:30 pm to 5:45 pm	0	0	0	0
	SBL	5:30 pm to 5:45 pm	10	0	0	10
	SBT	5:30 pm to 5:45 pm	14	13	0	27
	SBR	5:30 pm to 5:45 pm	7	0	0	7
	WBL	5:45 pm to 6:00 pm	0	1	0	1
	WBT	5:45 pm to 6:00 pm	7	0	0	7
	WBR	5:45 pm to 6:00 pm	8	0	0	8
	EBR	5:45 pm to 6:00 pm	0	0	0	0
	EBT	5:45 pm to 6:00 pm	8	0	0	8
	EBL	5:45 pm to 6:00 pm	2	0	0	2
	NBR	5:45 pm to 6:00 pm	2	0	0	2
	NBT	5:45 pm to 6:00 pm	13	12	0	25
	NBL	5:45 pm to 6:00 pm	0	0	0	0
	SBL	5:45 pm to 6:00 pm	9	0	0	9
	SBT	5:45 pm to 6:00 pm	11	19	0	30
	SBR	5:45 pm to 6:00 pm	1	0	0	1

A.1.2 OWSC at Cortland

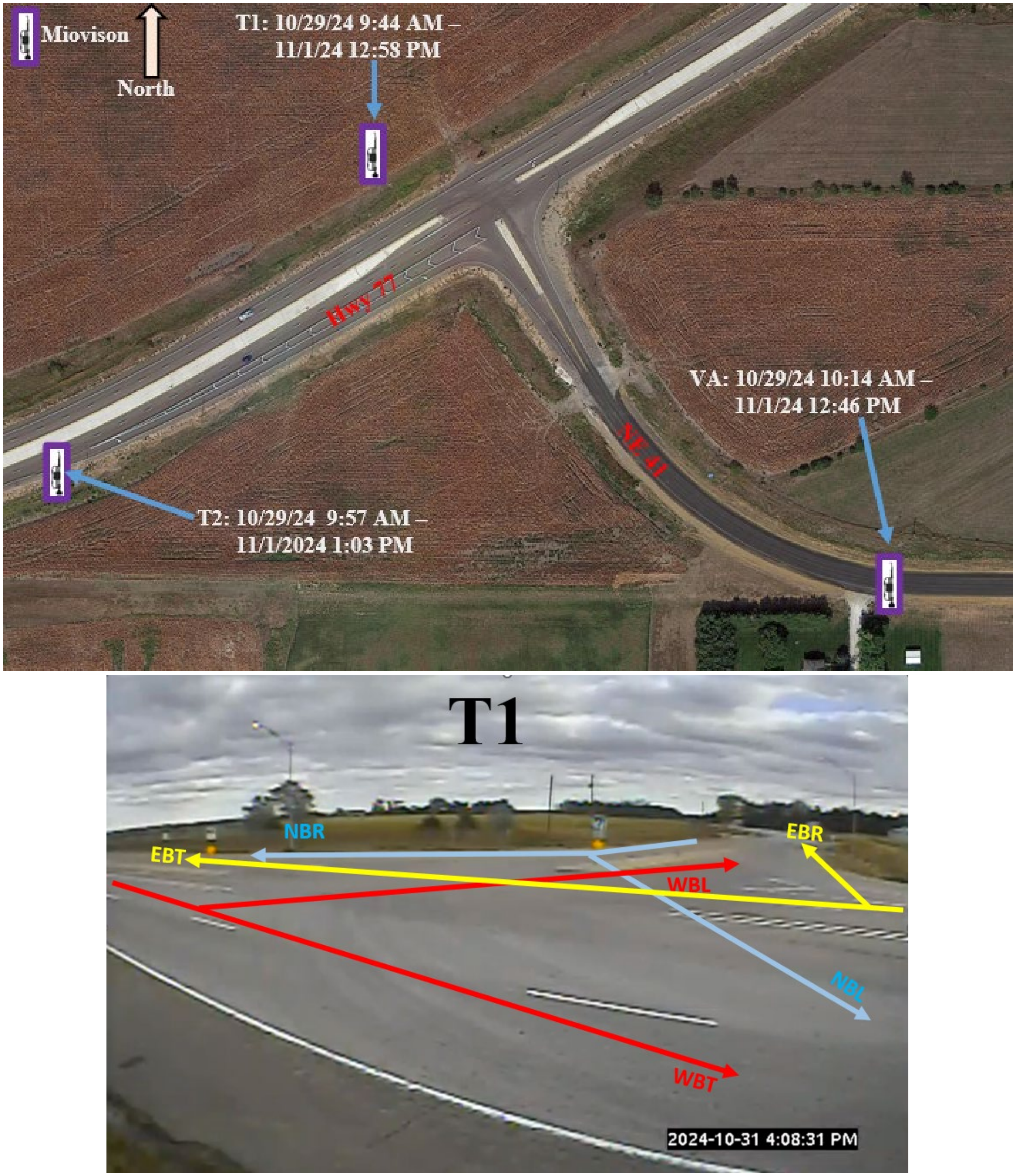


Figure A.2 Data collection device locations and corresponding traffic movements at the Cortland site

Table A.2 Traffic volume counts at the Cortland site

Recording Device	Movements	Time	Date	Vehicle Classification PC	TT	T	Total
Miovision: T1	NBL	7:00 am to 7:15 am	10/8/2024	51	6	6	63
	NBT	7:00 am to 7:15 am	10/8/2024	8	0	1	9
	NBR	7:00 am to 7:15 am	10/8/2024	103	6	1	110
	WBL	7:00 am to 7:15 am	10/8/2024	3	1	0	4
	WBT	7:00 am to 7:15 am	10/8/2024	13	0	0	13
	WBR	7:00 am to 7:15 am	10/8/2024	10	3	0	13
	NBL	7:15 am to 7:30 am	10/8/2024	67	10	1	78
	NBT	7:15 am to 7:30 am	10/8/2024	9	0	0	9
	NBR	7:15 am to 7:30 am	10/8/2024	117	3	0	120
	WBL	7:15 am to 7:30 am	10/8/2024	5	0	0	5
	WBT	7:15 am to 7:30 am	10/8/2024	12	2	0	14
	WBR	7:15 am to 7:30 am	10/8/2024	6	0	0	6
	NBL	7:30 am to 7:45 am	10/8/2024	89	3	2	94
	NBT	7:30 am to 7:45 am	10/8/2024	10	0	0	10
	NBR	7:30 am to 7:45 am	10/8/2024	90	3	1	94
	WBL	7:30 am to 7:45 am	10/8/2024	10	0	0	10
	WBT	7:30 am to 7:45 am	10/8/2024	15	0	0	15
	WBR	7:30 am to 7:45 am	10/8/2024	12	2	0	14
	NBL	7:45 am to 8:00 am	10/8/2024	79	5	5	89
	NBT	7:45 am to 8:00 am	10/8/2024	9	2	0	11
	NBR	7:45 am to 8:00 am	10/8/2024	73	4	3	80
	WBL	7:45 am to 8:00 am	10/8/2024	7	0	0	7
	WBT	7:45 am to 8:00 am	10/8/2024	16	0	0	16
	WBR	7:45 am to 8:00 am	10/8/2024	1	0	1	2
	NBL	8:00 am to 8:15 am	10/8/2024	58	1	3	62
	NBT	8:00 am to 8:15 am	10/8/2024	4	0	0	4
	NBR	8:00 am to 8:15 am	10/8/2024	77	10	2	89
	WBL	8:00 am to 8:15 am	10/8/2024	2	2	0	4
	WBT	8:00 am to 8:15 am	10/8/2024	13	3	0	16
	WBR	8:00 am to 8:15 am	10/8/2024	9	0	0	9
	NBL	8:15 am to 8:30 am	10/8/2024	49	3	6	58
	NBT	8:15 am to 8:30 am	10/8/2024	6	1	0	7
	NBR	8:15 am to 8:30 am	10/8/2024	82	4	0	86
	WBL	8:15 am to 8:30 am	10/8/2024	4	0	0	4
	WBT	8:15 am to 8:30 am	10/8/2024	11	0	0	11
	WBR	8:15 am to 8:30 am	10/8/2024	3	0	0	3
	NBL	8:30 am to 8:45 am	10/8/2024	58	7	2	67
	NBT	8:30 am to 8:45 am	10/8/2024	6	2	1	9
	NBR	8:30 am to 8:45 am	10/8/2024	65	2	2	69
	WBL	8:30 am to 8:45 am	10/8/2024	5	1	2	8
	WBT	8:30 am to 8:45 am	10/8/2024	3	1	0	4
	WBR	8:30 am to 8:45 am	10/8/2024	4	4	1	9
	NBL	8:45 am to 9:00 am	10/8/2024	55	3	4	62
	NBT	8:45 am to 9:00 am	10/8/2024	1	0	1	2
	NBR	8:45 am to 9:00 am	10/8/2024	63	10	2	75
	WBL	8:45 am to 9:00 am	10/8/2024	7	0	1	8
	WBT	8:45 am to 9:00 am	10/8/2024	1	1	0	2
	WBR	8:45 am to 9:00 am	10/8/2024	6	1	0	7

Recording Device	Movements	Time	Date	Vehicle Classification			
				PC	TT	T	Total
Miovision: T1	NBL	11:00 am to 11:15 am	10/9/2024	49	6	0	55
	NBT	11:00 am to 11:15 am	10/9/2024	2	1	0	3
	NBR	11:00 am to 11:15 am	10/9/2024	42	2	2	46
	WBL	11:00 am to 11:15 am	10/9/2024	5	1	0	6
	WBT	11:00 am to 11:15 am	10/9/2024	11	4	1	16
	WBR	11:00 am to 11:15 am	10/9/2024	11	2	0	13
	NBL	11:15 am to 11:30 am	10/9/2024	57	6	0	63
	NBT	11:15 am to 11:30 am	10/9/2024	8	0	0	8
	NBR	11:15 am to 11:30 am	10/9/2024	69	8	1	78
	WBL	11:15 am to 11:30 am	10/9/2024	3	2	1	6
	WBT	11:15 am to 11:30 am	10/9/2024	7	2	0	9
	WBR	11:15 am to 11:30 am	10/9/2024	8	1	0	9
	NBL	11:30 am to 11:45 am	10/9/2024	53	3	0	56
	NBT	11:30 am to 11:45 am	10/9/2024	5	1	0	6
	NBR	11:30 am to 11:45 am	10/9/2024	54	8	1	63
	WBL	11:30 am to 11:45 am	10/9/2024	4	2	0	6
	WBT	11:30 am to 11:45 am	10/9/2024	3	3	1	7
	WBR	11:30 am to 11:45 am	10/9/2024	4	0	0	4
	NBL	11:45 am to 12:00 pm	10/9/2024	56	2	2	60
	NBT	11:45 am to 12:00 pm	10/9/2024	2	2	0	4
	NBR	11:45 am to 12:00 pm	10/9/2024	73	8	4	85
	WBL	11:45 am to 12:00 pm	10/9/2024	3	4	0	7
	WBT	11:45 am to 12:00 pm	10/9/2024	2	0	1	3
	WBR	11:45 am to 12:00 pm	10/9/2024	10	0	1	11
	NBL	12:00 pm to 12:15 pm	10/9/2024	60	4	2	66
	NBT	12:00 pm to 12:15 pm	10/9/2024	6	0	0	6
	NBR	12:00 pm to 12:15 pm	10/9/2024	67	5	3	75
	WBL	12:00 pm to 12:15 pm	10/9/2024	3	0	0	3
	WBT	12:00 pm to 12:15 pm	10/9/2024	4	2	0	6
	WBR	12:00 pm to 12:15 pm	10/9/2024	4	1	1	6
	NBL	12:15 pm to 12:30 pm	10/9/2024	54	6	3	63
	NBT	12:15 pm to 12:30 pm	10/9/2024	6	0	1	7
	NBR	12:15 pm to 12:30 pm	10/9/2024	75	4	1	80
	WBL	12:15 pm to 12:30 pm	10/9/2024	4	2	0	6
	WBT	12:15 pm to 12:30 pm	10/9/2024	7	0	1	8
	WBR	12:15 pm to 12:30 pm	10/9/2024	2	2	0	4
	NBL	12:30 pm to 12:45 pm	10/9/2024	63	8	2	73
	NBT	12:30 pm to 12:45 pm	10/9/2024	6	1	0	7
	NBR	12:30 pm to 12:45 pm	10/9/2024	57	0	2	59
	WBL	12:30 pm to 12:45 pm	10/9/2024	5	2	1	8
	WBT	12:30 pm to 12:45 pm	10/9/2024	7	1	0	8
	WBR	12:30 pm to 12:45 pm	10/9/2024	11	1	0	12
	NBL	12:45 pm to 1:00 pm	10/9/2024	64	5	4	73
	NBT	12:45 pm to 1:00 pm	10/9/2024	10	3	0	13
	NBR	12:45 pm to 1:00 pm	10/9/2024	60	5	3	68
	WBL	12:45 pm to 1:00 pm	10/9/2024	5	2	1	8
	WBT	12:45 pm to 1:00 pm	10/9/2024	7	1	1	9
	WBR	12:45 pm to 1:00 pm	10/9/2024	4	1	0	5

Recording Device	Movements	Time	Date	Vehicle Classification			
				PC	TT	T	Total
Miovision: T1	NBL	4:00 pm to 4:15 pm	10/9/2024	107	4	2	113
	NBT	4:00 pm to 4:15 pm	10/9/2024	16	1	0	17
	NBR	4:00 pm to 4:15 pm	10/9/2024	86	6	2	94
	WBL	4:00 pm to 4:15 pm	10/9/2024	7	1	0	8
	WBT	4:00 pm to 4:15 pm	10/9/2024	5	1	0	6
	WBR	4:00 pm to 4:15 pm	10/9/2024	4	0	0	4
	NBL	4:15 pm to 4:30 pm	10/9/2024	106	5	0	111
	NBT	4:15 pm to 4:30 pm	10/9/2024	12	1	0	13
	NBR	4:15 pm to 4:30 pm	10/9/2024	95	4	0	99
	WBL	4:15 pm to 4:30 pm	10/9/2024	13	0	1	14
	WBT	4:15 pm to 4:30 pm	10/9/2024	14	0	0	14
	WBR	4:15 pm to 4:30 pm	10/9/2024	5	2	0	7
	NBL	4:30 pm to 4:45 pm	10/9/2024	129	5	1	135
	NBT	4:30 pm to 4:45 pm	10/9/2024	20	1	0	21
	NBR	4:30 pm to 4:45 pm	10/9/2024	73	2	2	77
	WBL	4:30 pm to 4:45 pm	10/9/2024	11	2	0	13
	WBT	4:30 pm to 4:45 pm	10/9/2024	11	0	0	11
	WBR	4:30 pm to 4:45 pm	10/9/2024	10	0	0	10
	NBL	4:45 pm to 5:00 pm	10/9/2024	132	9	4	145
	NBT	4:45 pm to 5:00 pm	10/9/2024	8	1	0	9
	NBR	4:45 pm to 5:00 pm	10/9/2024	86	3	1	90
	WBL	4:45 pm to 5:00 pm	10/9/2024	12	0	0	12
	WBT	4:45 pm to 5:00 pm	10/9/2024	17	1	0	18
	WBR	4:45 pm to 5:00 pm	10/9/2024	10	1	1	12
	NBL	5:00 pm to 5:15 pm	10/9/2024	115	4	2	121
	NBT	5:00 pm to 5:15 pm	10/9/2024	24	0	0	24
	NBR	5:00 pm to 5:15 pm	10/9/2024	57	2	1	60
	WBL	5:00 pm to 5:15 pm	10/9/2024	9	0	0	9
	WBT	5:00 pm to 5:15 pm	10/9/2024	21	0	0	21
	WBR	5:00 pm to 5:15 pm	10/9/2024	7	1	0	8
	NBL	5:15 pm to 5:30 pm	10/9/2024	138	3	2	143
	NBT	5:15 pm to 5:30 pm	10/9/2024	18	0	0	18
	NBR	5:15 pm to 5:30 pm	10/9/2024	87	3	3	93
	WBL	5:15 pm to 5:30 pm	10/9/2024	7	1	1	9
	WBT	5:15 pm to 5:30 pm	10/9/2024	5	1	0	6
	WBR	5:15 pm to 5:30 pm	10/9/2024	9	0	0	9
	NBL	5:30 pm to 5:45 pm	10/9/2024	117	3	1	121
	NBT	5:30 pm to 5:45 pm	10/9/2024	7	0	0	7
	NBR	5:30 pm to 5:45 pm	10/9/2024	69	2	2	73
	WBL	5:30 pm to 5:45 pm	10/9/2024	13	0	0	13
	WBT	5:30 pm to 5:45 pm	10/9/2024	5	0	0	5
	WBR	5:30 pm to 5:45 pm	10/9/2024	4	0	0	4
	NBL	5:45 pm to 6:00 pm	10/9/2024	69	3	0	72
	NBT	5:45 pm to 6:00 pm	10/9/2024	10	0	0	10
	NBR	5:45 pm to 6:00 pm	10/9/2024	66	4	0	70
	WBL	5:45 pm to 6:00 pm	10/9/2024	8	2	0	10
	WBT	5:45 pm to 6:00 pm	10/9/2024	6	2	0	8
	WBR	5:45 pm to 6:00 pm	10/9/2024	2	2	0	4

A.1.3 SI at Wahoo

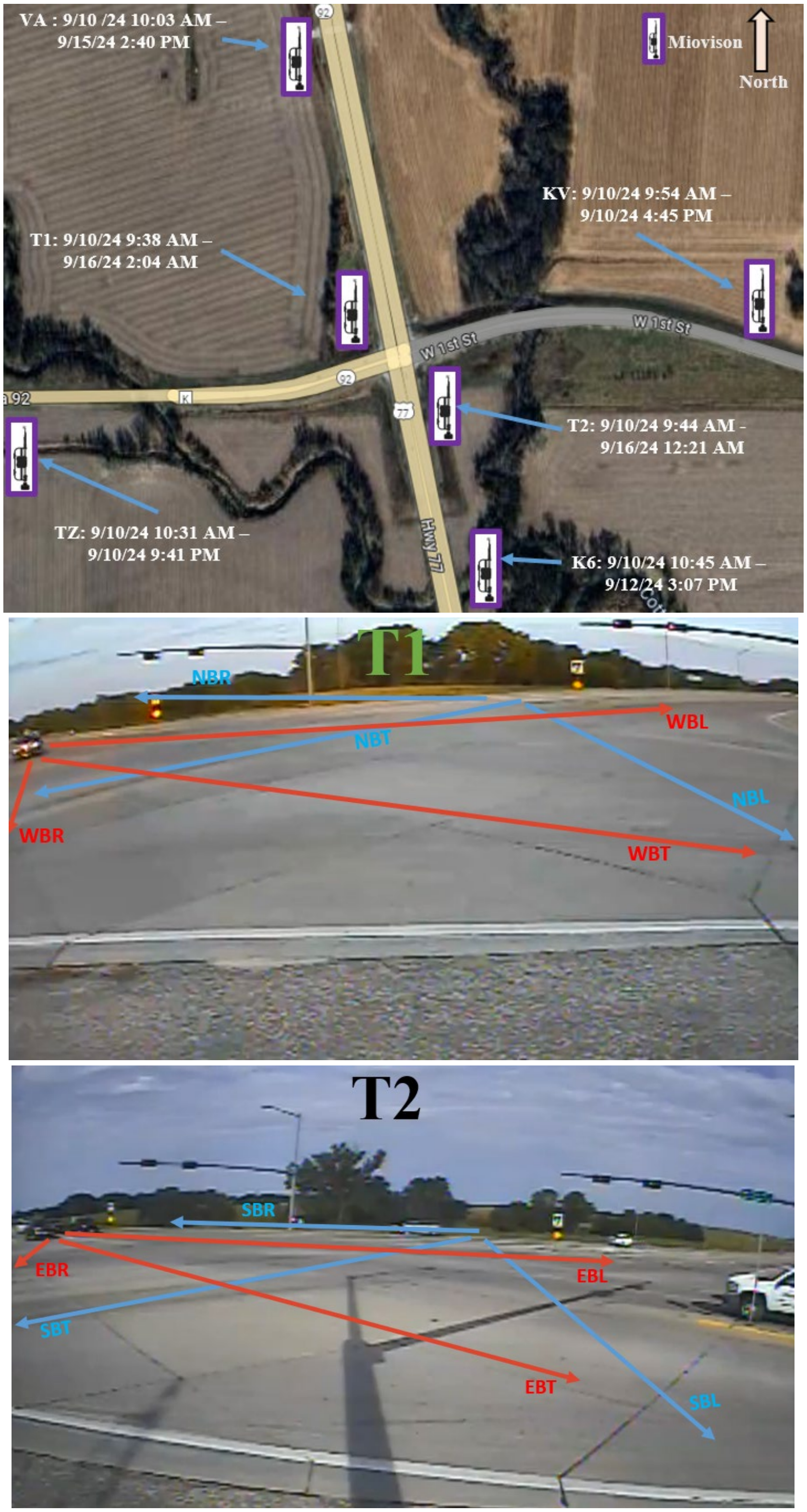


Figure A.3 Data collection device locations and corresponding traffic movements at the Wahoo site

Table A.3 Traffic volume counts at the Wahoo site

Date: 9/11/2024

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T1	SBL	8:00 am to 8:15 am	3	0	0	3
	SBT	8:00 am to 8:15 am	54	4	0	58
	SBR	8:00 am to 8:15 am	21	4	3	28
	EBL	8:00 am to 8:15 am	10	4	1	15
	EBT	8:00 am to 8:15 am	12	0	0	12
	EBR	8:00 am to 8:15 am	7	1	0	8
	SBL	8:15 am to 8:30 am	0	0	0	0
	SBT	8:15 am to 8:30 am	38	7	1	46
	SBR	8:15 am to 8:30 am	13	2	1	16
	EBL	8:15 am to 8:30 am	19	5	0	24
	EBT	8:15 am to 8:30 am	13	1	0	14
	EBR	8:15 am to 8:30 am	11	1	0	12
	SBL	8:30 am to 8:45 am	3	0	0	3
	SBT	8:30 am to 8:45 am	26	5	1	32
	SBR	8:30 am to 8:45 am	17	5	2	24
	EBL	8:30 am to 8:45 am	15	4	0	19
	EBT	8:30 am to 8:45 am	11	0	0	11
	EBR	8:30 am to 8:45 am	6	0	0	6
	SBL	8:45 am to 9:00 am	5	0	0	5
	SBT	8:45 am to 9:00 am	24	4	1	29
	SBR	8:45 am to 9:00 am	14	0	1	15
	EBL	8:45 am to 9:00 am	19	2	0	21
	EBT	8:45 am to 9:00 am	16	0	0	16
	EBR	8:45 am to 9:00 am	13	4	1	18
	SBL	9:00 am to 9:15 am	0	0	0	0
	SBT	9:00 am to 9:15 am	24	6	0	30
	SBR	9:00 am to 9:15 am	18	2	4	24
	EBL	9:00 am to 9:15 am	19	6	0	25
	EBT	9:00 am to 9:15 am	8	1	0	9
	EBR	9:00 am to 9:15 am	9	1	0	10
	SBL	9:15 am to 9:30 am	0	0	0	0
	SBT	9:15 am to 9:30 am	31	9	0	40
	SBR	9:15 am to 9:30 am	21	3	0	24
	EBL	9:15 am to 9:30 am	10	7	3	20
	EBT	9:15 am to 9:30 am	9	0	0	9
	EBR	9:15 am to 9:30 am	13	3	0	16
	SBL	9:30 am to 9:45 am	0	0	0	0
	SBT	9:30 am to 9:45 am	33	4	3	40
	SBR	9:30 am to 9:45 am	7	5	0	12
	EBL	9:30 am to 9:45 am	18	4	0	22
	EBT	9:30 am to 9:45 am	14	0	0	14
	EBR	9:30 am to 9:45 am	12	4	1	17
	SBL	9:45 am to 10:00 am	0	1	0	1
	SBT	9:45 am to 10:00 am	24	5	0	29
	SBR	9:45 am to 10:00 am	10	0	1	11
	EBL	9:45 am to 10:00 am	6	3	2	11
	EBT	9:45 am to 10:00 am	12	0	0	12
	EBR	9:45 am to 10:00 am	5	1	1	7

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T1	SBL	3:00 pm to 3:15 pm	0	0	0	0
	SBT	3:00 pm to 3:15 pm	32	3	0	35
	SBR	3:00 pm to 3:15 pm	16	1	1	18
	EBL	3:00 pm to 3:15 pm	19	4	1	24
	EBT	3:00 pm to 3:15 pm	20	0	0	20
	EBR	3:00 pm to 3:15 pm	10	2	0	12
	SBL	3:15 pm to 3:30 pm	6	0	0	6
	SBT	3:15 pm to 3:30 pm	41	8	1	50
	SBR	3:15 pm to 3:30 pm	14	1	0	15
	EBL	3:15 pm to 3:30 pm	20	2	0	22
	EBT	3:15 pm to 3:30 pm	15	2	2	19
	EBR	3:15 pm to 3:30 pm	6	5	0	11
	SBL	3:30 pm to 3:45 pm	4	0	0	4
	SBT	3:30 pm to 3:45 pm	61	4	1	66
	SBR	3:30 pm to 3:45 pm	20	0	1	21
	EBL	3:30 pm to 3:45 pm	24	2	0	26
	EBT	3:30 pm to 3:45 pm	11	0	0	11
	EBR	3:30 pm to 3:45 pm	10	4	0	14
	SBL	3:45 pm to 4:00 pm	5	0	0	5
	SBT	3:45 pm to 4:00 pm	56	7	2	65
	SBR	3:45 pm to 4:00 pm	15	2	0	17
	EBL	3:45 pm to 4:00 pm	28	1	0	29
	EBT	3:45 pm to 4:00 pm	9	0	0	9
	EBR	3:45 pm to 4:00 pm	15	0	1	16
	SBL	4:00 pm to 4:15 pm	1	0	1	2
	SBT	4:00 pm to 4:15 pm	57	9	2	68
	SBR	4:00 pm to 4:15 pm	21	4	0	25
	EBL	4:00 pm to 4:15 pm	22	6	1	29
	EBT	4:00 pm to 4:15 pm	11	0	2	13
	EBR	4:00 pm to 4:15 pm	16	0	1	17
	SBL	4:15 pm to 4:30 pm	2	0	0	2
	SBT	4:15 pm to 4:30 pm	24	3	1	28
	SBR	4:15 pm to 4:30 pm	26	2	0	28
	EBL	4:15 pm to 4:30 pm	17	3	0	20
	EBT	4:15 pm to 4:30 pm	10	0	0	10
	EBR	4:15 pm to 4:30 pm	8	3	0	11
	SBL	4:30 pm to 4:45 pm	4	0	0	4
	SBT	4:30 pm to 4:45 pm	68	7	8	83
	SBR	4:30 pm to 4:45 pm	19	0	0	19
	EBL	4:30 pm to 4:45 pm	25	1	0	26
	EBT	4:30 pm to 4:45 pm	14	0	1	15
	EBR	4:30 pm to 4:45 pm	14	3	0	17
	SBL	4:45 pm to 5:00 pm	4	0	0	4
	SBT	4:45 pm to 5:00 pm	46	10	1	57
	SBR	4:45 pm to 5:00 pm	32	2	0	34
	EBL	4:45 pm to 5:00 pm	26	4	0	30
	EBT	4:45 pm to 5:00 pm	16	0	0	16
	EBR	4:45 pm to 5:00 pm	13	3	0	16

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	NBR	8:00 am to 8:15 am	7	3	2	12
	NBT	8:00 am to 8:15 am	40	3	1	44
	NBL	8:00 am to 8:15 am	10	0	0	10
	WBR	8:00 am to 8:15 am	21	0	0	21
	WBT	8:00 am to 8:15 am	6	0	0	6
	WBL	8:00 am to 8:15 am	8	0	0	8
	NBR	8:15 am to 8:30 am	17	4	0	21
	NBT	8:15 am to 8:30 am	41	2	1	44
	NBL	8:15 am to 8:30 am	10	0	0	10
	WBR	8:15 am to 8:30 am	24	0	1	25
	WBT	8:15 am to 8:30 am	7	0	0	7
	WBL	8:15 am to 8:30 am	2	0	0	2
	NBR	8:30 am to 8:45 am	8	1	0	9
	NBT	8:30 am to 8:45 am	28	13	2	43
	NBL	8:30 am to 8:45 am	8	0	1	9
	WBR	8:30 am to 8:45 am	20	0	0	20
	WBT	8:30 am to 8:45 am	11	0	1	12
	WBL	8:30 am to 8:45 am	1	0	0	1
	NBR	8:45 am to 9:00 am	6	3	2	11
	NBT	8:45 am to 9:00 am	26	7	3	36
	NBL	8:45 am to 9:00 am	4	0	0	4
	WBR	8:45 am to 9:00 am	15	0	0	15
	WBT	8:45 am to 9:00 am	14	0	0	14
	WBL	8:45 am to 9:00 am	4	0	0	4
	NBR	9:00 am to 9:15 am	10	1	1	12
	NBT	9:00 am to 9:15 am	23	8	0	31
	NBL	9:00 am to 9:15 am	11	1	1	13
	WBR	9:00 am to 9:15 am	12	0	0	12
	WBT	9:00 am to 9:15 am	8	0	1	9
	WBL	9:00 am to 9:15 am	1	0	0	1
	NBR	9:15 am to 9:30 am	9	5	6	20
	NBT	9:15 am to 9:30 am	39	5	0	44
	NBL	9:15 am to 9:30 am	8	0	1	9
	WBR	9:15 am to 9:30 am	9	0	0	9
	WBT	9:15 am to 9:30 am	9	0	0	9
	WBL	9:15 am to 9:30 am	1	0	0	1
	NBR	9:30 am to 9:45 am	7	4	1	12
	NBT	9:30 am to 9:45 am	30	4	3	37
	NBL	9:30 am to 9:45 am	9	0	0	9
	WBR	9:30 am to 9:45 am	8	0	0	8
	WBT	9:30 am to 9:45 am	12	0	0	12
	WBL	9:30 am to 9:45 am	5	0	0	5
	NBR	9:45 am to 10:00 am	6	3	0	9
	NBT	9:45 am to 10:00 am	21	6	0	27
	NBL	9:45 am to 10:00 am	9	0	0	9
	WBR	9:45 am to 10:00 am	8	0	1	9
	WBT	9:45 am to 10:00 am	8	0	0	8
	WBL	9:45 am to 10:00 am	6	0	0	6

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	NBR	3:00 pm to 3:15 pm	11	1	1	13
	NBT	3:00 pm to 3:15 pm	36	2	3	41
	NBL	3:00 pm to 3:15 pm	17	0	0	17
	WBR	3:00 pm to 3:15 pm	10	0	1	11
	WBT	3:00 pm to 3:15 pm	15	2	0	17
	WBL	3:00 pm to 3:15 pm	2	0	0	2
	NBR	3:15 pm to 3:30 pm	7	0	0	7
	NBT	3:15 pm to 3:30 pm	45	4	0	49
	NBL	3:15 pm to 3:30 pm	18	0	0	18
	WBR	3:15 pm to 3:30 pm	9	0	0	9
	WBT	3:15 pm to 3:30 pm	18	0	1	19
	WBL	3:15 pm to 3:30 pm	2	0	0	2
	NBR	3:30 pm to 3:45 pm	15	0	0	15
	NBT	3:30 pm to 3:45 pm	43	4	0	47
	NBL	3:30 pm to 3:45 pm	14	0	0	14
	WBR	3:30 pm to 3:45 pm	39	0	0	39
	WBT	3:30 pm to 3:45 pm	36	1	0	37
	WBL	3:30 pm to 3:45 pm	11	0	0	11
	NBR	3:45 pm to 4:00 pm	12	2	0	14
	NBT	3:45 pm to 4:00 pm	46	4	1	51
	NBL	3:45 pm to 4:00 pm	28	0	0	28
	WBR	3:45 pm to 4:00 pm	25	0	0	25
	WBT	3:45 pm to 4:00 pm	24	0	0	24
	WBL	3:45 pm to 4:00 pm	9	0	0	9
	NBR	4:00 pm to 4:15 pm	13	2	0	15
	NBT	4:00 pm to 4:15 pm	54	7	0	61
	NBL	4:00 pm to 4:15 pm	30	0	0	30
	WBR	4:00 pm to 4:15 pm	19	0	0	19
	WBT	4:00 pm to 4:15 pm	17	0	0	17
	WBL	4:00 pm to 4:15 pm	5	0	0	5
	NBR	4:15 pm to 4:30 pm	12	5	0	17
	NBT	4:15 pm to 4:30 pm	42	4	0	46
	NBL	4:15 pm to 4:30 pm	17	0	0	17
	WBR	4:15 pm to 4:30 pm	15	0	0	15
	WBT	4:15 pm to 4:30 pm	24	0	0	24
	WBL	4:15 pm to 4:30 pm	4	0	0	4
	NBR	4:30 pm to 4:45 pm	17	0	1	18
	NBT	4:30 pm to 4:45 pm	61	3	0	64
	NBL	4:30 pm to 4:45 pm	28	0	0	28
	WBR	4:30 pm to 4:45 pm	24	0	1	25
	WBT	4:30 pm to 4:45 pm	25	0	0	25
	WBL	4:30 pm to 4:45 pm	7	0	0	7
	NBR	4:45 pm to 5:00 pm	19	2	0	21
	NBT	4:45 pm to 5:00 pm	47	3	0	50
	NBL	4:45 pm to 5:00 pm	36	0	1	37
	WBR	4:45 pm to 5:00 pm	14	0	0	14
	WBT	4:45 pm to 5:00 pm	23	0	0	23
	WBL	4:45 pm to 5:00 pm	3	0	0	3

Date: 9/11/2024

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T1	SBL	12:00 pm to 12:15 pm	2	0	0	2
	SBT	12:00 pm to 12:15 pm	31	9	2	42
	SBR	12:00 pm to 12:15 pm	8	4	1	13
	EBL	12:00 pm to 12:15 pm	12	3	0	15
	EBT	12:00 pm to 12:15 pm	10	0	0	10
	EBR	12:00 pm to 12:15 pm	8	3	1	12
	SBL	12:15 pm to 12:30 pm	2	0	0	2
	SBT	12:15 pm to 12:30 pm	23	1	1	25
	SBR	12:15 pm to 12:30 pm	9	1	1	11
	EBL	12:15 pm to 12:30 pm	7	6	6	19
	EBT	12:15 pm to 12:30 pm	7	0	0	7
	EBR	12:15 pm to 12:30 pm	8	5	5	18
	SBL	12:30 pm to 12:45 pm	4	0	0	4
	SBT	12:30 pm to 12:45 pm	26	13	2	41
	SBR	12:30 pm to 12:45 pm	16	5	0	21
	EBL	12:30 pm to 12:45 pm	13	3	0	16
	EBT	12:30 pm to 12:45 pm	9	0	1	10
	EBR	12:30 pm to 12:45 pm	5	0	1	6
	SBL	12:45 pm to 1:00 pm	2	0	0	2
	SBT	12:45 pm to 1:00 pm	20	7	0	27
	SBR	12:45 pm to 1:00 pm	9	4	3	16
	EBL	12:45 pm to 1:00 pm	9	4	2	15
	EBT	12:45 pm to 1:00 pm	7	0	1	8
	EBR	12:45 pm to 1:00 pm	7	3	0	10
	SBL	1:00 pm to 1:15 pm	1	0	0	1
	SBT	1:00 pm to 1:15 pm	28	8	4	40
	SBR	1:00 pm to 1:15 pm	25	5	0	30
	EBL	1:00 pm to 1:15 pm	7	5	1	13
	EBT	1:00 pm to 1:15 pm	7	0	0	7
	EBR	1:00 pm to 1:15 pm	10	3	0	13
	SBL	1:15 pm to 1:30 pm	0	0	0	0
	SBT	1:15 pm to 1:30 pm	28	4	0	32
	SBR	1:15 pm to 1:30 pm	10	4	1	15
	EBL	1:15 pm to 1:30 pm	10	3	2	15
	EBT	1:15 pm to 1:30 pm	7	1	0	8
	EBR	1:15 pm to 1:30 pm	8	0	0	8
	SBL	1:30 pm to 1:45 pm	1	0	0	1
	SBT	1:30 pm to 1:45 pm	34	8	2	44
	SBR	1:30 pm to 1:45 pm	17	3	0	20
	EBL	1:30 pm to 1:45 pm	7	0	1	8
	EBT	1:30 pm to 1:45 pm	8	0	1	9
	EBR	1:30 pm to 1:45 pm	15	6	0	21
	SBL	1:45 pm to 2:00 pm	3	0	0	3
	SBT	1:45 pm to 2:00 pm	36	4	3	43
	SBR	1:45 pm to 2:00 pm	11	3	2	16
	EBL	1:45 pm to 2:00 pm	11	1	1	13
	EBT	1:45 pm to 2:00 pm	7	0	2	9
	EBR	1:45 pm to 2:00 pm	5	4	1	10

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T1	SBL	4:00 pm to 4:15 pm	4	0	0	4
	SBT	4:00 pm to 4:15 pm	53	2	1	56
	SBR	4:00 pm to 4:15 pm	19	2	0	21
	EBL	4:00 pm to 4:15 pm	23	2	1	26
	EBT	4:00 pm to 4:15 pm	24	1	2	27
	EBR	4:00 pm to 4:15 pm	10	4	0	14
	SBL	4:15 pm to 4:30 pm	9	0	0	9
	SBT	4:15 pm to 4:30 pm	66	8	3	77
	SBR	4:15 pm to 4:30 pm	18	2	1	21
	EBL	4:15 pm to 4:30 pm	16	4	0	20
	EBT	4:15 pm to 4:30 pm	19	0	1	20
	EBR	4:15 pm to 4:30 pm	11	5	1	17
	SBL	4:30 pm to 4:45 pm	4	0	0	4
	SBT	4:30 pm to 4:45 pm	51	6	1	58
	SBR	4:30 pm to 4:45 pm	31	0	0	31
	EBL	4:30 pm to 4:45 pm	26	1	1	28
	EBT	4:30 pm to 4:45 pm	17	0	1	18
	EBR	4:30 pm to 4:45 pm	20	0	2	22
	SBL	4:45 pm to 5:00 pm	3	0	0	3
	SBT	4:45 pm to 5:00 pm	53	9	2	64
	SBR	4:45 pm to 5:00 pm	26	0	0	26
	EBL	4:45 pm to 5:00 pm	25	1	0	26
	EBT	4:45 pm to 5:00 pm	16	2	0	18
	EBR	4:45 pm to 5:00 pm	14	0	0	14
	SBL	5:00 pm to 5:15 pm	8	0	0	8
	SBT	5:00 pm to 5:15 pm	83	4	0	87
	SBR	5:00 pm to 5:15 pm	21	2	0	23
	EBL	5:00 pm to 5:15 pm	22	1	0	23
	EBT	5:00 pm to 5:15 pm	19	0	0	19
	EBR	5:00 pm to 5:15 pm	14	0	0	14
	SBL	5:15 pm to 5:30 pm	11	0	0	11
	SBT	5:15 pm to 5:30 pm	57	3	1	61
	SBR	5:15 pm to 5:30 pm	20	0	1	21
	EBL	5:15 pm to 5:30 pm	24	2	0	26
	EBT	5:15 pm to 5:30 pm	17	0	1	18
	EBR	5:15 pm to 5:30 pm	13	4	0	17
	SBL	5:30 pm to 5:45 pm	5	0	0	5
	SBT	5:30 pm to 5:45 pm	69	3	0	72
	SBR	5:30 pm to 5:45 pm	23	0	1	24
	EBL	5:30 pm to 5:45 pm	20	3	0	23
	EBT	5:30 pm to 5:45 pm	12	0	1	13
	EBR	5:30 pm to 5:45 pm	12	2	0	14
	SBL	5:45 pm to 6:00 pm	6	0	0	6
	SBT	5:45 pm to 6:00 pm	61	0	1	62
	SBR	5:45 pm to 6:00 pm	25	1	0	26
	EBL	5:45 pm to 6:00 pm	35	1	1	37
	EBT	5:45 pm to 6:00 pm	25	0	0	25
	EBR	5:45 pm to 6:00 pm	6	1	1	8

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	NBR	12:00 pm to 12:15 pm	4	5	2	11
	NBT	12:00 pm to 12:15 pm	25	12	2	39
	NBL	12:00 pm to 12:15 pm	17	0	0	17
	WBR	12:00 pm to 12:15 pm	14	0	0	14
	WBT	12:00 pm to 12:15 pm	13	0	0	13
	WBL	12:00 pm to 12:15 pm	3	0	0	3
	NBR	12:15 pm to 12:30 pm	5	3	0	8
	NBT	12:15 pm to 12:30 pm	23	7	2	32
	NBL	12:15 pm to 12:30 pm	12	2	0	14
	WBR	12:15 pm to 12:30 pm	23	0	1	24
	WBT	12:15 pm to 12:30 pm	14	0	1	15
	WBL	12:15 pm to 12:30 pm	1	0	0	1
	NBR	12:30 pm to 12:45 pm	11	4	0	15
	NBT	12:30 pm to 12:45 pm	27	5	1	33
	NBL	12:30 pm to 12:45 pm	9	1	0	10
	WBR	12:30 pm to 12:45 pm	13	1	0	14
	WBT	12:30 pm to 12:45 pm	6	0	0	6
	WBL	12:30 pm to 12:45 pm	2	1	0	3
	NBR	12:45 pm to 1:00 pm	4	2	1	7
	NBT	12:45 pm to 1:00 pm	21	6	4	31
	NBL	12:45 pm to 1:00 pm	15	1	0	16
	WBR	12:45 pm to 1:00 pm	11	0	0	11
	WBT	12:45 pm to 1:00 pm	13	0	1	14
	WBL	12:45 pm to 1:00 pm	0	0	0	0
	NBR	1:00 pm to 1:15 pm	17	3	0	20
	NBT	1:00 pm to 1:15 pm	28	6	3	37
	NBL	1:00 pm to 1:15 pm	11	0	0	11
	WBR	1:00 pm to 1:15 pm	10	0	0	10
	WBT	1:00 pm to 1:15 pm	9	1	2	12
	WBL	1:00 pm to 1:15 pm	3	1	0	4
	NBR	1:15 pm to 1:30 pm	7	3	1	11
	NBT	1:15 pm to 1:30 pm	28	2	3	33
	NBL	1:15 pm to 1:30 pm	7	0	0	7
	WBR	1:15 pm to 1:30 pm	13	1	1	15
	WBT	1:15 pm to 1:30 pm	6	0	0	6
	WBL	1:15 pm to 1:30 pm	3	0	0	3
	NBR	1:30 pm to 1:45 pm	5	5	1	11
	NBT	1:30 pm to 1:45 pm	29	9	1	39
	NBL	1:30 pm to 1:45 pm	18	0	0	18
	WBR	1:30 pm to 1:45 pm	15	0	0	15
	WBT	1:30 pm to 1:45 pm	11	0	0	11
	WBL	1:30 pm to 1:45 pm	2	0	0	2
	NBR	1:45 pm to 2:00 pm	9	2	0	11
	NBT	1:45 pm to 2:00 pm	33	4	1	38
	NBL	1:45 pm to 2:00 pm	19	0	0	19
	WBR	1:45 pm to 2:00 pm	13	0	0	13
	WBT	1:45 pm to 2:00 pm	15	1	0	16
	WBL	1:45 pm to 2:00 pm	4	0	0	4

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	NBR	4:00 pm to 4:15 pm	13	2	0	15
	NBT	4:00 pm to 4:15 pm	56	3	1	60
	NBL	4:00 pm to 4:15 pm	29	0	0	29
	WBR	4:00 pm to 4:15 pm	23	0	0	23
	WBT	4:00 pm to 4:15 pm	21	0	0	21
	WBL	4:00 pm to 4:15 pm	3	0	0	3
	NBR	4:15 pm to 4:30 pm	16	3	0	19
	NBT	4:15 pm to 4:30 pm	55	5	2	62
	NBL	4:15 pm to 4:30 pm	35	0	1	36
	WBR	4:15 pm to 4:30 pm	14	0	0	14
	WBT	4:15 pm to 4:30 pm	23	0	0	23
	WBL	4:15 pm to 4:30 pm	4	0	0	4
	NBR	4:30 pm to 4:45 pm	8	0	0	8
	NBT	4:30 pm to 4:45 pm	74	1	1	76
	NBL	4:30 pm to 4:45 pm	43	1	0	44
	WBR	4:30 pm to 4:45 pm	18	0	1	19
	WBT	4:30 pm to 4:45 pm	28	0	0	28
	WBL	4:30 pm to 4:45 pm	6	0	0	6
	NBR	4:45 pm to 5:00 pm	18	1	0	19
	NBT	4:45 pm to 5:00 pm	65	4	1	70
	NBL	4:45 pm to 5:00 pm	38	0	0	38
	WBR	4:45 pm to 5:00 pm	13	0	1	14
	WBT	4:45 pm to 5:00 pm	27	0	0	27
	WBL	4:45 pm to 5:00 pm	2	0	0	2
	NBR	5:00 pm to 5:15 pm	16	1	1	18
	NBT	5:00 pm to 5:15 pm	70	1	0	71
	NBL	5:00 pm to 5:15 pm	33	0	0	33
	WBR	5:00 pm to 5:15 pm	19	0	0	19
	WBT	5:00 pm to 5:15 pm	27	0	0	27
	WBL	5:00 pm to 5:15 pm	3	1	0	4
	NBR	5:15 pm to 5:30 pm	16	1	0	17
	NBT	5:15 pm to 5:30 pm	52	2	1	55
	NBL	5:15 pm to 5:30 pm	28	0	0	28
	WBR	5:15 pm to 5:30 pm	11	0	0	11
	WBT	5:15 pm to 5:30 pm	21	0	0	21
	WBL	5:15 pm to 5:30 pm	2	0	0	2
	NBR	5:30 pm to 5:45 pm	15	2	0	17
	NBT	5:30 pm to 5:45 pm	56	6	2	64
	NBL	5:30 pm to 5:45 pm	28	0	0	28
	WBR	5:30 pm to 5:45 pm	25	0	0	25
	WBT	5:30 pm to 5:45 pm	18	0	0	18
	WBL	5:30 pm to 5:45 pm	6	0	0	6
	NBR	5:45 pm to 6:00 pm	16	1	0	17
	NBT	5:45 pm to 6:00 pm	47	2	0	49
	NBL	5:45 pm to 6:00 pm	20	0	1	21
	WBR	5:45 pm to 6:00 pm	18	0	0	18
	WBT	5:45 pm to 6:00 pm	31	0	0	31
	WBL	5:45 pm to 6:00 pm	7	0	0	7

A.1.4 SI at Waverly

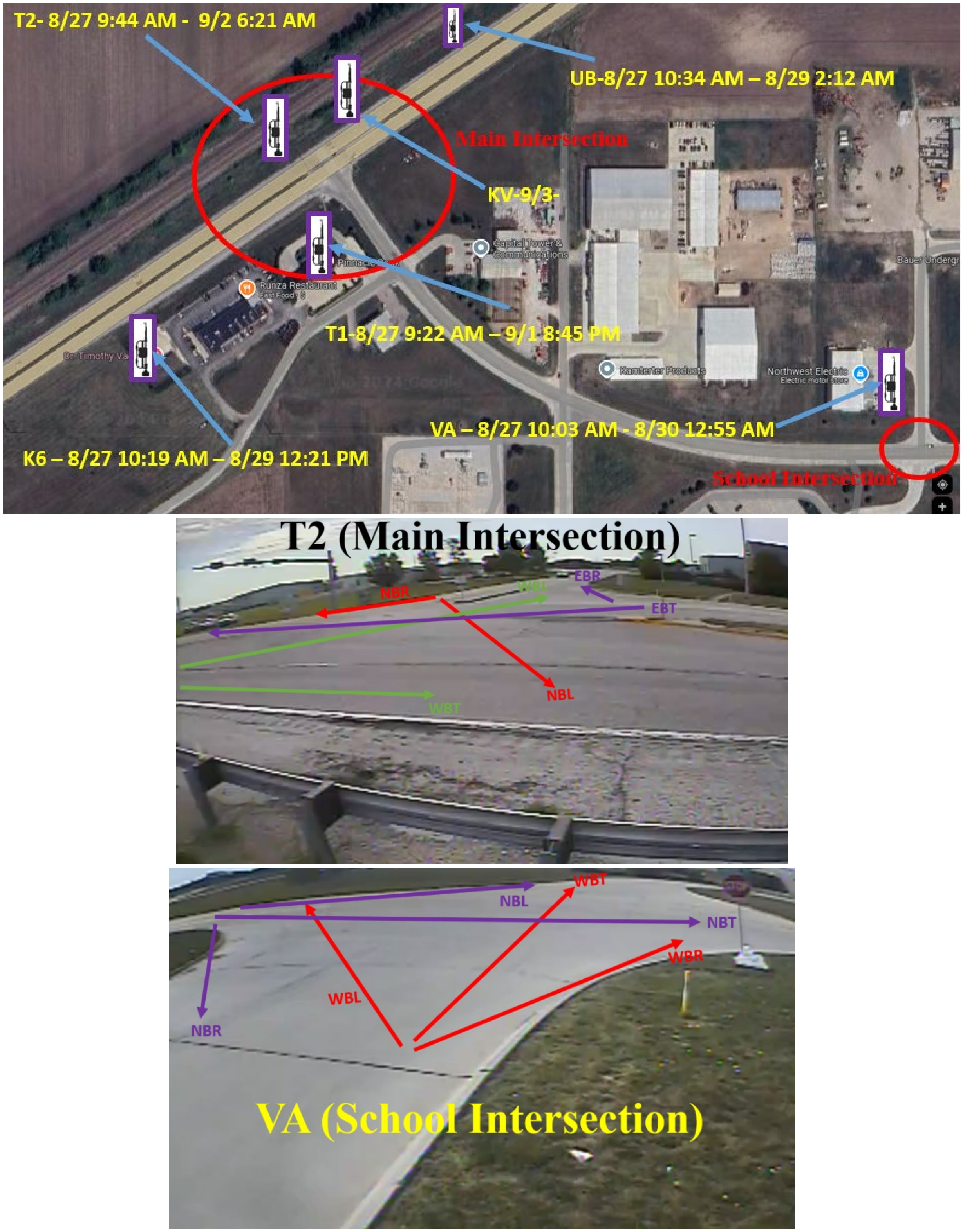


Figure A.4 Data collection device locations and corresponding traffic movements at the Waverly site

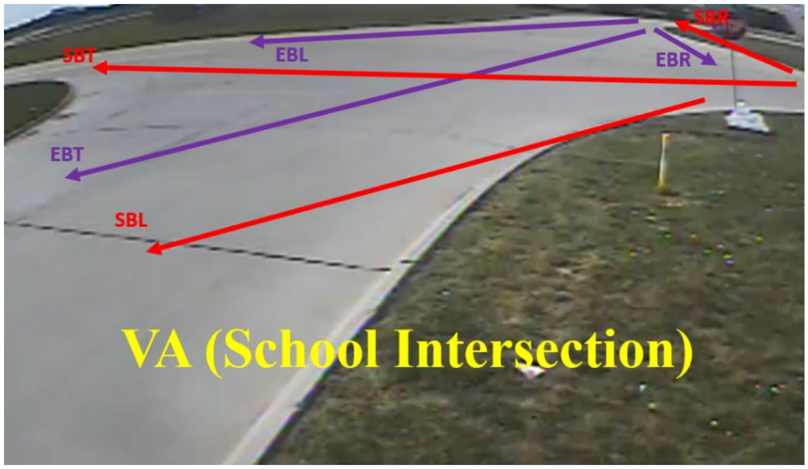


Figure A.4 cont. Data collection device locations and corresponding traffic movements at the Waverly site

Table A.4 Traffic volume counts at the Waverly site

Date: 8/28/2024

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	WBT	7:00 am to 7:15 am	81	8	1	90
	WBL	7:00 am to 7:15 am	4	0	0	4
	NBR	7:00 am to 7:15 am	37	8	4	49
	NBT	7:00 am to 7:15 am	16	2	1	19
	EBR	7:00 am to 7:15 am	67	7	0	74
	EBT	7:00 am to 7:15 am	2	0	1	3
	WBT	7:15 am to 7:30 am	83	12	1	96
	WBL	7:15 am to 7:30 am	18	0	0	18
	NBR	7:15 am to 7:30 am	64	7	3	74
	NBT	7:15 am to 7:30 am	23	2	0	25
	EBR	7:15 am to 7:30 am	51	7	1	59
	EBT	7:15 am to 7:30 am	2	0	0	2
	WBT	7:30 am to 7:45 am	119	10	2	131
	WBL	7:30 am to 7:45 am	10	0	0	10
	NBR	7:30 am to 7:45 am	64	5	5	74
	NBT	7:30 am to 7:45 am	32	2	0	34
	EBR	7:30 am to 7:45 am	58	3	1	62
	EBT	7:30 am to 7:45 am	4	0	0	4
	WBT	7:45 am to 8:00 am	84	7	3	94
	WBL	7:45 am to 8:00 am	10	0	0	10
	NBR	7:45 am to 8:00 am	76	6	3	85
	NBT	7:45 am to 8:00 am	65	1	1	67
	EBR	7:45 am to 8:00 am	42	2	0	44
	EBT	7:45 am to 8:00 am	0	0	0	0

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	WBT	8:00 am to 8:15 am	126	6	2	134
	WBL	8:00 am to 8:15 am	17	0	0	17
	NBR	8:00 am to 8:15 am	50	7	2	59
	NBT	8:00 am to 8:15 am	31	3	0	34
	EBR	8:00 am to 8:15 am	43	4	0	47
	EBT	8:00 am to 8:15 am	8	0	0	8
	WBT	8:15 am to 8:30 am	95	8	4	107
	WBL	8:15 am to 8:30 am	21	0	0	21
	NBR	8:15 am to 8:30 am	44	7	4	55
	NBT	8:15 am to 8:30 am	26	1	0	27
	EBR	8:15 am to 8:30 am	53	3	1	57
	EBT	8:15 am to 8:30 am	12	0	0	12
	WBT	8:30 am to 8:45 am	67	16	2	85
	WBL	8:30 am to 8:45 am	33	0	0	33
	NBR	8:30 am to 8:45 am	51	8	4	63
	NBT	8:30 am to 8:45 am	9	2	0	11
	EBR	8:30 am to 8:45 am	19	3	1	23
	EBT	8:30 am to 8:45 am	5	0	0	5
	WBT	8:45 am to 9:00 am	54	8	2	64
	WBL	8:45 am to 9:00 am	15	0	0	15
	NBR	8:45 am to 9:00 am	57	5	2	64
	NBT	8:45 am to 9:00 am	21	3	0	24
	EBR	8:45 am to 9:00 am	14	4	0	18
	EBT	8:45 am to 9:00 am	3	0	0	3
	WBT	9:00 am to 9:15 am	50	10	2	62
	WBL	9:00 am to 9:15 am	4	0	0	4
	NBR	9:00 am to 9:15 am	40	6	3	49
	NBT	9:00 am to 9:15 am	12	3	0	15
	EBR	9:00 am to 9:15 am	21	0	0	21
	EBT	9:00 am to 9:15 am	4	0	0	4
	WBT	9:15 am to 9:30 am	61	12	1	74
	WBL	9:15 am to 9:30 am	2	0	0	2
	NBR	9:15 am to 9:30 am	40	3	3	46
	NBT	9:15 am to 9:30 am	16	2	0	18
	EBR	9:15 am to 9:30 am	17	3	0	20
	EBT	9:15 am to 9:30 am	1	0	0	1
	WBT	9:30 am to 9:45 am	49	8	5	62
	WBL	9:30 am to 9:45 am	6	0	0	6
	NBR	9:30 am to 9:45 am	50	9	1	60
	NBT	9:30 am to 9:45 am	10	5	0	15
	EBR	9:30 am to 9:45 am	24	2	0	26
	EBT	9:30 am to 9:45 am	1	0	0	1
	WBT	9:45 am to 10:00 am	49	11	2	62
	WBL	9:45 am to 10:00 am	2	1	0	3
	NBR	9:45 am to 10:00 am	35	5	3	43
	NBT	9:45 am to 10:00 am	16	3	0	19
	EBR	9:45 am to 10:00 am	10	1	1	12
	EBT	9:45 am to 10:00 am	4	0	0	4

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	WBT	3:00 pm to 3:15 pm	43	9	3	55
	WBL	3:00 pm to 3:15 pm	5	1	0	6
	NBR	3:00 pm to 3:15 pm	96	5	0	101
	NBT	3:00 pm to 3:15 pm	49	2	0	51
	EBR	3:00 pm to 3:15 pm	19	1	0	20
	EBT	3:00 pm to 3:15 pm	4	0	0	4
	WBT	3:15 pm to 3:30 pm	69	6	4	79
	WBL	3:15 pm to 3:30 pm	10	0	0	10
	NBR	3:15 pm to 3:30 pm	73	2	1	76
	NBT	3:15 pm to 3:30 pm	53	2	0	55
	EBR	3:15 pm to 3:30 pm	20	2	0	22
	EBT	3:15 pm to 3:30 pm	6	0	0	6
	WBT	3:30 pm to 3:45 pm	77	6	0	83
	WBL	3:30 pm to 3:45 pm	5	0	0	5
	NBR	3:30 pm to 3:45 pm	80	4	0	84
	NBT	3:30 pm to 3:45 pm	36	2	0	38
	EBR	3:30 pm to 3:45 pm	28	1	0	29
	EBT	3:30 pm to 3:45 pm	7	0	0	7
	WBT	3:45 pm to 4:00 pm	79	7	1	87
	WBL	3:45 pm to 4:00 pm	7	0	0	7
	NBR	3:45 pm to 4:00 pm	111	2	0	113
	NBT	3:45 pm to 4:00 pm	35	3	0	38
	EBR	3:45 pm to 4:00 pm	42	0	0	42
	EBT	3:45 pm to 4:00 pm	35	0	0	35
	WBT	4:00 pm to 4:15 pm	108	2	3	113
	WBL	4:00 pm to 4:15 pm	11	0	0	11
	NBR	4:00 pm to 4:15 pm	88	6	0	94
	NBT	4:00 pm to 4:15 pm	45	1	1	47
	EBR	4:00 pm to 4:15 pm	56	2	1	59
	EBT	4:00 pm to 4:15 pm	8	0	0	8
	WBT	4:15 pm to 4:30 pm	89	5	2	96
	WBL	4:15 pm to 4:30 pm	3	0	0	3
	NBR	4:15 pm to 4:30 pm	103	7	1	111
	NBT	4:15 pm to 4:30 pm	69	1	1	71
	EBR	4:15 pm to 4:30 pm	31	3	0	34
	EBT	4:15 pm to 4:30 pm	8	0	0	8
	WBT	4:30 pm to 4:45 pm	84	4	1	89
	WBL	4:30 pm to 4:45 pm	5	0	0	5
	NBR	4:30 pm to 4:45 pm	100	2	5	107
	NBT	4:30 pm to 4:45 pm	53	1	1	55
	EBR	4:30 pm to 4:45 pm	32	2	1	35
	EBT	4:30 pm to 4:45 pm	10	0	0	10
	WBT	4:45 pm to 5:00 pm	69	0	0	69
	WBL	4:45 pm to 5:00 pm	3	0	0	3
	NBR	4:45 pm to 5:00 pm	89	2	2	93
	NBT	4:45 pm to 5:00 pm	61	2	0	63
	EBR	4:45 pm to 5:00 pm	43	1	0	44
	EBT	4:45 pm to 5:00 pm	5	0	0	5

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	WBT	5:00 pm to 5:15 pm	75	3	0	78
	WBL	5:00 pm to 5:15 pm	5	0	0	5
	NBR	5:00 pm to 5:15 pm	126	6	3	135
	NBT	5:00 pm to 5:15 pm	49	1	1	51
	EBR	5:00 pm to 5:15 pm	49	7	0	56
	EBT	5:00 pm to 5:15 pm	4	0	0	4
	WBT	5:15 pm to 5:30 pm	69	2	1	72
	WBL	5:15 pm to 5:30 pm	6	0	0	6
	NBR	5:15 pm to 5:30 pm	116	2	0	118
	NBT	5:15 pm to 5:30 pm	77	2	0	79
	EBR	5:15 pm to 5:30 pm	29	3	1	33
	EBT	5:15 pm to 5:30 pm	3	0	0	3
	WBT	5:30 pm to 5:45 pm	76	1	2	79
	WBL	5:30 pm to 5:45 pm	6	0	0	6
	NBR	5:30 pm to 5:45 pm	81	1	0	82
	NBT	5:30 pm to 5:45 pm	48	2	0	50
	EBR	5:30 pm to 5:45 pm	31	2	0	33
	EBT	5:30 pm to 5:45 pm	7	0	0	7
	WBT	5:45 pm to 6:00 pm	66	2	2	70
	WBL	5:45 pm to 6:00 pm	4	0	0	4
	NBR	5:45 pm to 6:00 pm	79	0	1	80
	NBT	5:45 pm to 6:00 pm	33	1	0	34
	EBR	5:45 pm to 6:00 pm	31	0	0	31
	EBT	5:45 pm to 6:00 pm	5	0	0	5

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
VA	NBR	7:00 am to 7:15 am	0	0	0	0
	NBT	7:00 am to 7:15 am	74	8	1	83
	NBL	7:00 am to 7:15 am	14	0	0	14
	WBL	7:00 am to 7:15 am	2	0	0	2
	WBT	7:00 am to 7:15 am	0	0	0	0
	WBR	7:00 am to 7:15 am	4	0	0	4
	SBR	7:00 am to 7:15 am	4	0	0	4
	SBT	7:00 am to 7:15 am	12	3	0	15
	SBL	7:00 am to 7:15 am	1	0	0	1
	EBL	7:00 am to 7:15 am	1	0	0	1
	EBT	7:00 am to 7:15 am	0	0	0	0
	EBR	7:00 am to 7:15 am	3	0	0	3
	NBR	7:15 am to 7:30 am	0	0	0	0
	NBT	7:15 am to 7:30 am	57	6	1	64
	NBL	7:15 am to 7:30 am	29	0	0	29
	WBL	7:15 am to 7:30 am	2	0	0	2
	WBT	7:15 am to 7:30 am	0	0	0	0
	WBR	7:15 am to 7:30 am	2	1	0	3
	SBR	7:15 am to 7:30 am	6	0	0	6
	SBT	7:15 am to 7:30 am	21	3	0	24
	SBL	7:15 am to 7:30 am	0	0	0	0
	EBL	7:15 am to 7:30 am	4	0	0	4
	EBT	7:15 am to 7:30 am	0	0	0	0
	EBR	7:15 am to 7:30 am	10	0	0	10
	NBR	7:30 am to 7:45 am	2	0	0	2
	NBT	7:30 am to 7:45 am	63	2	0	65
	NBL	7:30 am to 7:45 am	15	0	0	15
	WBL	7:30 am to 7:45 am	0	0	0	0
	WBT	7:30 am to 7:45 am	0	0	0	0
	WBR	7:30 am to 7:45 am	1	0	0	1
	SBR	7:30 am to 7:45 am	7	0	0	7
	SBT	7:30 am to 7:45 am	26	2	0	28
	SBL	7:30 am to 7:45 am	1	0	0	1
	EBL	7:30 am to 7:45 am	2	0	0	2
	EBT	7:30 am to 7:45 am	0	0	0	0
	EBR	7:30 am to 7:45 am	6	0	0	6
	NBR	7:45 am to 8:00 am	0	0	0	0
	NBT	7:45 am to 8:00 am	57	2	0	59
	NBL	7:45 am to 8:00 am	18	0	0	18
	WBL	7:45 am to 8:00 am	0	0	0	0
	WBT	7:45 am to 8:00 am	0	0	0	0
	WBR	7:45 am to 8:00 am	1	0	0	1
	SBR	7:45 am to 8:00 am	9	0	0	9
	SBT	7:45 am to 8:00 am	41	2	2	45
	SBL	7:45 am to 8:00 am	3	0	0	3
	EBL	7:45 am to 8:00 am	2	0	0	2
	EBT	7:45 am to 8:00 am	0	0	0	0
	EBR	7:45 am to 8:00 am	3	0	0	3

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
VA	NBR	8:00 am to 8:15 am	1	0	0	1
	NBT	8:00 am to 8:15 am	56	4	0	60
	NBL	8:00 am to 8:15 am	12	0	0	12
	WBL	8:00 am to 8:15 am	7	0	0	7
	WBT	8:00 am to 8:15 am	0	0	0	0
	WBR	8:00 am to 8:15 am	0	0	0	0
	SBR	8:00 am to 8:15 am	3	0	0	3
	SBT	8:00 am to 8:15 am	54	3	1	58
	SBL	8:00 am to 8:15 am	1	0	0	1
	EBL	8:00 am to 8:15 am	1	0	0	1
	EBT	8:00 am to 8:15 am	0	0	0	0
	EBR	8:00 am to 8:15 am	6	0	0	6
	NBR	8:15 am to 8:30 am	0	0	0	0
	NBT	8:15 am to 8:30 am	80	4	1	85
	NBL	8:15 am to 8:30 am	11	0	0	11
	WBL	8:15 am to 8:30 am	1	0	0	1
	WBT	8:15 am to 8:30 am	0	0	0	0
	WBR	8:15 am to 8:30 am	0	0	0	0
	SBR	8:15 am to 8:30 am	7	0	0	7
	SBT	8:15 am to 8:30 am	52	1	0	53
	SBL	8:15 am to 8:30 am	0	0	0	0
	EBL	8:15 am to 8:30 am	6	0	0	6
	EBT	8:15 am to 8:30 am	0	0	0	0
	EBR	8:15 am to 8:30 am	6	0	0	6
	NBR	8:30 am to 8:45 am	0	0	0	0
	NBT	8:30 am to 8:45 am	54	4	2	60
	NBL	8:30 am to 8:45 am	12	0	0	12
	WBL	8:30 am to 8:45 am	1	0	0	1
	WBT	8:30 am to 8:45 am	0	0	0	0
	WBR	8:30 am to 8:45 am	1	0	0	1
	SBR	8:30 am to 8:45 am	3	0	0	3
	SBT	8:30 am to 8:45 am	15	2	0	17
	SBL	8:30 am to 8:45 am	1	0	0	1
	EBL	8:30 am to 8:45 am	2	0	0	2
	EBT	8:30 am to 8:45 am	0	0	0	0
	EBR	8:30 am to 8:45 am	7	0	0	7
	NBR	8:45 am to 9:00 am	0	0	0	0
	NBT	8:45 am to 9:00 am	30	3	0	33
	NBL	8:45 am to 9:00 am	20	0	0	20
	WBL	8:45 am to 9:00 am	0	0	0	0
	WBT	8:45 am to 9:00 am	0	0	0	0
	WBR	8:45 am to 9:00 am	2	0	0	2
	SBR	8:45 am to 9:00 am	3	0	0	3
	SBT	8:45 am to 9:00 am	26	3	1	30
	SBL	8:45 am to 9:00 am	1	0	0	1
	EBL	8:45 am to 9:00 am	4	0	0	4
	EBT	8:45 am to 9:00 am	0	0	0	0
	EBR	8:45 am to 9:00 am	17	0	0	17

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
VA	NBR	9:00 am to 9:15 am	0	0	0	0
	NBT	9:00 am to 9:15 am	19	0	0	19
	NBL	9:00 am to 9:15 am	3	0	0	3
	WBL	9:00 am to 9:15 am	1	0	0	1
	WBT	9:00 am to 9:15 am	0	0	0	0
	WBR	9:00 am to 9:15 am	1	0	0	1
	SBR	9:00 am to 9:15 am	1	0	0	1
	SBT	9:00 am to 9:15 am	19	3	0	22
	SBL	9:00 am to 9:15 am	1	0	0	1
	EBL	9:00 am to 9:15 am	2	0	0	2
	EBT	9:00 am to 9:15 am	0	0	0	0
	EBR	9:00 am to 9:15 am	2	0	0	2
	NBR	9:15 am to 9:30 am	0	0	0	0
	NBT	9:15 am to 9:30 am	22	3	0	25
	NBL	9:15 am to 9:30 am	0	0	0	0
	WBL	9:15 am to 9:30 am	0	0	0	0
	WBT	9:15 am to 9:30 am	0	0	0	0
	WBR	9:15 am to 9:30 am	2	0	0	2
	SBR	9:15 am to 9:30 am	0	0	0	0
	SBT	9:15 am to 9:30 am	18	2	0	20
	SBL	9:15 am to 9:30 am	0	0	0	0
	EBL	9:15 am to 9:30 am	1	0	0	1
	EBT	9:15 am to 9:30 am	0	0	0	0
	EBR	9:15 am to 9:30 am	2	0	0	2
	NBR	9:30 am to 9:45 am	0	0	0	0
	NBT	9:30 am to 9:45 am	24	3	0	27
	NBL	9:30 am to 9:45 am	3	0	0	3
	WBL	9:30 am to 9:45 am	0	0	0	0
	WBT	9:30 am to 9:45 am	0	0	0	0
	WBR	9:30 am to 9:45 am	0	0	0	0
	SBR	9:30 am to 9:45 am	0	0	0	0
	SBT	9:30 am to 9:45 am	11	5	0	16
	SBL	9:30 am to 9:45 am	0	0	0	0
	EBL	9:30 am to 9:45 am	1	0	0	1
	EBT	9:30 am to 9:45 am	0	0	0	0
	EBR	9:30 am to 9:45 am	1	0	0	1
	NBR	9:45 am to 10:00 am	2	0	0	2
	NBT	9:45 am to 10:00 am	11	6	1	18
	NBL	9:45 am to 10:00 am	2	0	0	2
	WBL	9:45 am to 10:00 am	0	0	0	0
	WBT	9:45 am to 10:00 am	0	0	0	0
	WBR	9:45 am to 10:00 am	1	0	0	1
	SBR	9:45 am to 10:00 am	0	0	0	0
	SBT	9:45 am to 10:00 am	14	4	0	18
	SBL	9:45 am to 10:00 am	2	0	0	2
	EBL	9:45 am to 10:00 am	0	0	0	0
	EBT	9:45 am to 10:00 am	0	0	0	0
	EBR	9:45 am to 10:00 am	1	0	0	1

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
VA	NBR	3:00 pm to 3:15 pm	0	0	0	0
	NBT	3:00 pm to 3:15 pm	16	1	0	17
	NBL	3:00 pm to 3:15 pm	1	0	0	1
	WBL	3:00 pm to 3:15 pm	1	0	0	1
	WBT	3:00 pm to 3:15 pm	0	0	0	0
	WBR	3:00 pm to 3:15 pm	0	0	0	0
	SBR	3:00 pm to 3:15 pm	1	0	0	1
	SBT	3:00 pm to 3:15 pm	48	3	0	51
	SBL	3:00 pm to 3:15 pm	4	0	0	4
	EBL	3:00 pm to 3:15 pm	3	0	0	3
	EBT	3:00 pm to 3:15 pm	0	0	0	0
	EBR	3:00 pm to 3:15 pm	0	0	0	0
	NBR	3:15 pm to 3:30 pm	0	0	0	0
	NBT	3:15 pm to 3:30 pm	21	2	0	23
	NBL	3:15 pm to 3:30 pm	9	0	0	9
	WBL	3:15 pm to 3:30 pm	0	0	0	0
	WBT	3:15 pm to 3:30 pm	0	0	0	0
	WBR	3:15 pm to 3:30 pm	0	0	0	0
	SBR	3:15 pm to 3:30 pm	3	0	0	3
	SBT	3:15 pm to 3:30 pm	32	3	1	36
	SBL	3:15 pm to 3:30 pm	4	0	0	4
	EBL	3:15 pm to 3:30 pm	1	0	0	1
	EBT	3:15 pm to 3:30 pm	0	0	0	0
	EBR	3:15 pm to 3:30 pm	0	0	0	0
	NBR	3:30 pm to 3:45 pm	0	0	0	0
	NBT	3:30 pm to 3:45 pm	35	1	0	36
	NBL	3:30 pm to 3:45 pm	19	0	0	19
	WBL	3:30 pm to 3:45 pm	0	0	0	0
	WBT	3:30 pm to 3:45 pm	0	0	0	0
	WBR	3:30 pm to 3:45 pm	1	0	0	1
	SBR	3:30 pm to 3:45 pm	3	0	0	3
	SBT	3:30 pm to 3:45 pm	52	2	0	54
	SBL	3:30 pm to 3:45 pm	0	0	0	0
	EBL	3:30 pm to 3:45 pm	3	0	0	3
	EBT	3:30 pm to 3:45 pm	0	0	0	0
	EBR	3:30 pm to 3:45 pm	17	0	0	17
	NBR	3:45 pm to 4:00 pm	0	0	0	0
	NBT	3:45 pm to 4:00 pm	27	1	0	28
	NBL	3:45 pm to 4:00 pm	7	0	0	7
	WBL	3:45 pm to 4:00 pm	0	0	0	0
	WBT	3:45 pm to 4:00 pm	0	0	0	0
	WBR	3:45 pm to 4:00 pm	1	0	0	1
	SBR	3:45 pm to 4:00 pm	4	0	0	4
	SBT	3:45 pm to 4:00 pm	135	3	0	138
	SBL	3:45 pm to 4:00 pm	0	0	0	0
	EBL	3:45 pm to 4:00 pm	3	0	0	3
	EBT	3:45 pm to 4:00 pm	0	0	0	0
	EBR	3:45 pm to 4:00 pm	36	0	0	36

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
VA	NBR	4:00 pm to 4:15 pm	0	0	0	0
	NBT	4:00 pm to 4:15 pm	43	2	1	46
	NBL	4:00 pm to 4:15 pm	4	0	0	4
	WBL	4:00 pm to 4:15 pm	0	0	0	0
	WBT	4:00 pm to 4:15 pm	0	0	0	0
	WBR	4:00 pm to 4:15 pm	1	0	0	1
	SBR	4:00 pm to 4:15 pm	3	0	0	3
	SBT	4:00 pm to 4:15 pm	64	1	2	67
	SBL	4:00 pm to 4:15 pm	0	0	0	0
	EBL	4:00 pm to 4:15 pm	10	0	0	10
	EBT	4:00 pm to 4:15 pm	0	0	0	0
	EBR	4:00 pm to 4:15 pm	19	0	0	19
	NBR	4:15 pm to 4:30 pm	0	0	0	0
	NBT	4:15 pm to 4:30 pm	24	4	0	28
	NBL	4:15 pm to 4:30 pm	1	0	0	1
	WBL	4:15 pm to 4:30 pm	1	0	0	1
	WBT	4:15 pm to 4:30 pm	0	0	0	0
	WBR	4:15 pm to 4:30 pm	1	0	0	1
	SBR	4:15 pm to 4:30 pm	2	0	0	2
	SBT	4:15 pm to 4:30 pm	83	1	1	85
	SBL	4:15 pm to 4:30 pm	1	0	0	1
	EBL	4:15 pm to 4:30 pm	3	0	0	3
	EBT	4:15 pm to 4:30 pm	0	0	0	0
	EBR	4:15 pm to 4:30 pm	2	0	0	2
	NBR	4:30 pm to 4:45 pm	0	0	0	0
	NBT	4:30 pm to 4:45 pm	38	2	1	41
	NBL	4:30 pm to 4:45 pm	2	0	0	2
	WBL	4:30 pm to 4:45 pm	1	0	0	1
	WBT	4:30 pm to 4:45 pm	0	0	0	0
	WBR	4:30 pm to 4:45 pm	4	0	0	4
	SBR	4:30 pm to 4:45 pm	2	0	0	2
	SBT	4:30 pm to 4:45 pm	56	1	1	58
	SBL	4:30 pm to 4:45 pm	0	0	0	0
	EBL	4:30 pm to 4:45 pm	5	0	0	5
	EBT	4:30 pm to 4:45 pm	0	0	0	0
	EBR	4:30 pm to 4:45 pm	8	0	0	8
	NBR	4:45 pm to 5:00 pm	0	0	0	0
	NBT	4:45 pm to 5:00 pm	34	4	2	40
	NBL	4:45 pm to 5:00 pm	2	0	0	2
	WBL	4:45 pm to 5:00 pm	0	0	0	0
	WBT	4:45 pm to 5:00 pm	0	0	0	0
	WBR	4:45 pm to 5:00 pm	0	0	0	0
	SBR	4:45 pm to 5:00 pm	3	0	0	3
	SBT	4:45 pm to 5:00 pm	79	1	1	81
	SBL	4:45 pm to 5:00 pm	0	0	0	0
	EBL	4:45 pm to 5:00 pm	2	0	0	2
	EBT	4:45 pm to 5:00 pm	0	0	0	0
	EBR	4:45 pm to 5:00 pm	3	0	0	3

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
VA	NBR	5:00 pm to 5:15 pm	0	0	0	0
	NBT	5:00 pm to 5:15 pm	38	2	0	40
	NBL	5:00 pm to 5:15 pm	1	0	0	1
	WBL	5:00 pm to 5:15 pm	0	0	0	0
	WBT	5:00 pm to 5:15 pm	0	0	0	0
	WBR	5:00 pm to 5:15 pm	0	0	0	0
	SBR	5:00 pm to 5:15 pm	0	0	0	0
	SBT	5:00 pm to 5:15 pm	70	2	1	73
	SBL	5:00 pm to 5:15 pm	1	0	0	1
	EBL	5:00 pm to 5:15 pm	1	0	0	1
	EBT	5:00 pm to 5:15 pm	0	0	0	0
	EBR	5:00 pm to 5:15 pm	3	0	0	3
	NBR	5:15 pm to 5:30 pm	0	0	0	0
	NBT	5:15 pm to 5:30 pm	20	3	1	24
	NBL	5:15 pm to 5:30 pm	0	0	0	0
	WBL	5:15 pm to 5:30 pm	1	0	0	1
	WBT	5:15 pm to 5:30 pm	0	0	0	0
	WBR	5:15 pm to 5:30 pm	2	0	0	2
	SBR	5:15 pm to 5:30 pm	3	0	0	3
	SBT	5:15 pm to 5:30 pm	77	1	0	78
	SBL	5:15 pm to 5:30 pm	6	1	0	7
	EBL	5:15 pm to 5:30 pm	0	0	0	0
	EBT	5:15 pm to 5:30 pm	0	0	0	0
	EBR	5:15 pm to 5:30 pm	3	0	0	3
	NBR	5:30 pm to 5:45 pm	0	0	0	0
	NBT	5:30 pm to 5:45 pm	35	2	0	37
	NBL	5:30 pm to 5:45 pm	2	0	0	2
	WBL	5:30 pm to 5:45 pm	1	0	0	1
	WBT	5:30 pm to 5:45 pm	0	0	0	0
	WBR	5:30 pm to 5:45 pm	3	0	0	3
	SBR	5:30 pm to 5:45 pm	0	0	0	0
	SBT	5:30 pm to 5:45 pm	70	2	0	72
	SBL	5:30 pm to 5:45 pm	0	0	0	0
	EBL	5:30 pm to 5:45 pm	1	0	0	1
	EBT	5:30 pm to 5:45 pm	0	0	0	0
	EBR	5:30 pm to 5:45 pm	2	0	0	2
	NBR	5:45 pm to 6:00 pm	0	0	0	0
	NBT	5:45 pm to 6:00 pm	26	1	0	27
	NBL	5:45 pm to 6:00 pm	5	0	0	5
	WBL	5:45 pm to 6:00 pm	0	0	0	0
	WBT	5:45 pm to 6:00 pm	0	0	0	0
	WBR	5:45 pm to 6:00 pm	1	0	0	1
	SBR	5:45 pm to 6:00 pm	1	0	0	1
	SBT	5:45 pm to 6:00 pm	41	1	0	42
	SBL	5:45 pm to 6:00 pm	0	0	0	0
	EBL	5:45 pm to 6:00 pm	1	0	0	1
	EBT	5:45 pm to 6:00 pm	0	0	0	0
	EBR	5:45 pm to 6:00 pm	1	0	0	1

A.1.5 RCUT at Humphrey

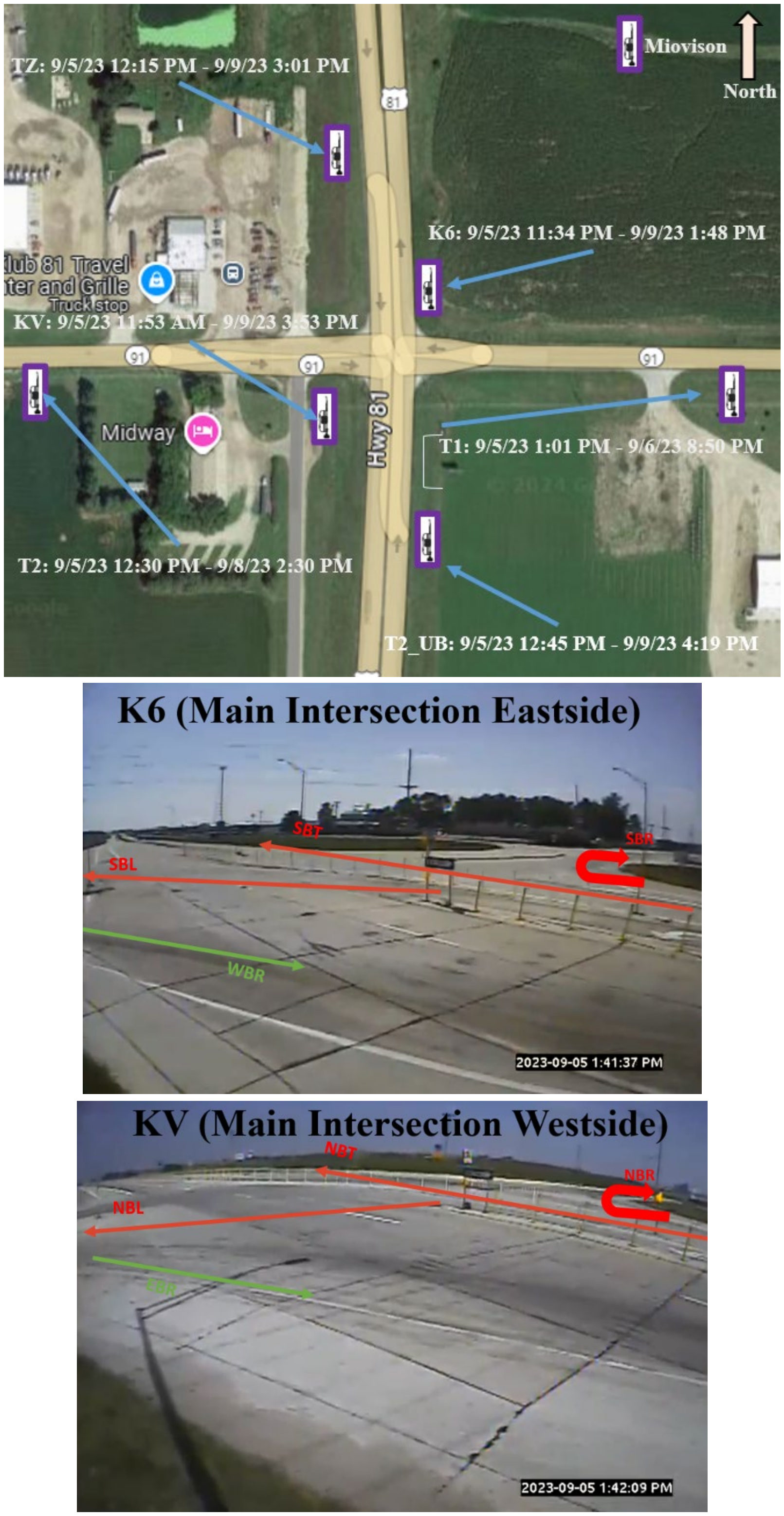
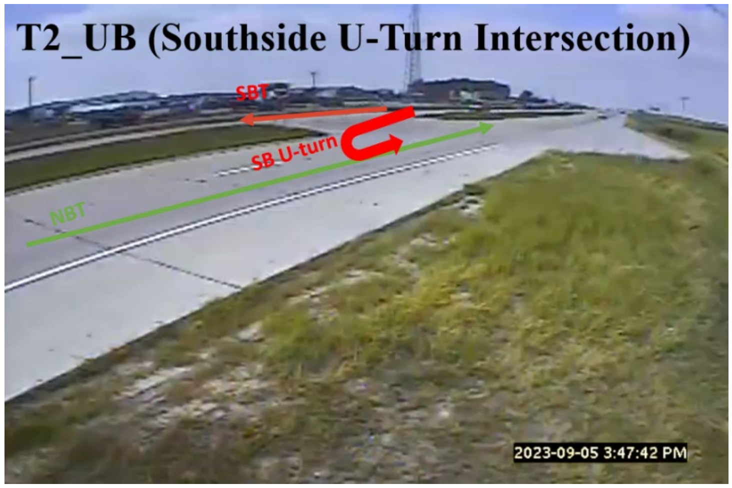


Figure A.5 Data collection device locations and corresponding traffic movements at the Humphrey site



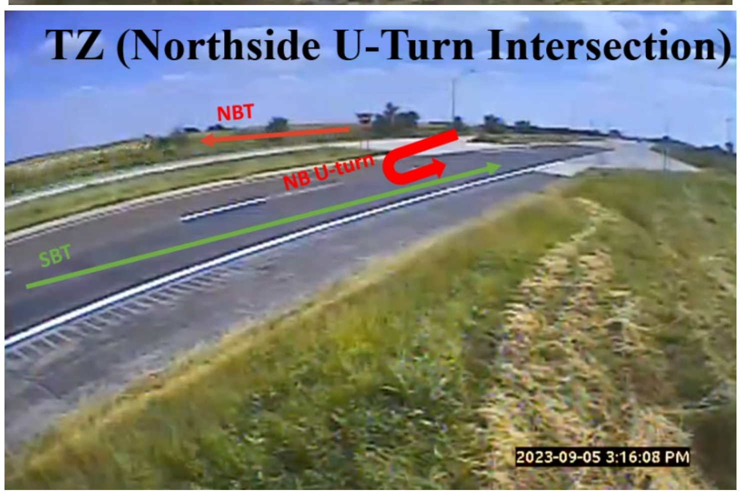


Figure A.5 cont. Data collection device locations and corresponding traffic movements at the Humphrey site

Table A.5 Traffic volume counts at the Humphrey site

Date: 9/6/2023

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: K6	WBR	8:00 am to 8:15 am	14	1	0	15
	SBL	8:00 am to 8:15 am	1	0	0	1
	SBT	8:00 am to 8:15 am	31	10	2	43
	SBR	8:00 am to 8:15 am	18	2	0	20
	WBR	8:15 am to 8:30 am	10	0	0	10
	SBL	8:15 am to 8:30 am	5	2	0	7
	SBT	8:15 am to 8:30 am	33	8	2	43
	SBR	8:15 am to 8:30 am	11	1	0	12
	WBR	8:30 am to 8:45 am	12	5	0	17
	SBL	8:30 am to 8:45 am	0	2	0	2
	SBT	8:30 am to 8:45 am	32	9	1	42
	SBR	8:30 am to 8:45 am	15	2	1	18
	WBR	8:45 am to 9:00 am	11	3	1	15
	SBL	8:45 am to 9:00 am	1	0	1	2
	SBT	8:45 am to 9:00 am	29	9	2	40
	SBR	8:45 am to 9:00 am	16	4	0	20
	WBR	9:00 am to 9:15 am	12	1	0	13
	SBL	9:00 am to 9:15 am	2	1	0	3
	SBT	9:00 am to 9:15 am	30	11	1	42
	SBR	9:00 am to 9:15 am	14	3	0	17
	WBR	9:15 am to 9:30 am	15	5	0	20
	SBL	9:15 am to 9:30 am	1	0	0	1
	SBT	9:15 am to 9:30 am	32	9	1	42
	SBR	9:15 am to 9:30 am	12	4	0	16
	WBR	9:30 am to 9:45 am	11	8	0	19
	SBL	9:30 am to 9:45 am	3	0	0	3
	SBT	9:30 am to 9:45 am	30	9	0	39
	SBR	9:30 am to 9:45 am	20	5	0	25
	WBR	9:45 am to 10:00 am	9	2	0	11
	SBL	9:45 am to 10:00 am	4	2	0	6
	SBT	9:45 am to 10:00 am	29	15	2	46
	SBR	9:45 am to 10:00 am	8	1	1	10

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: K6	WBR	4:00 pm to 4:15 pm	8	7	0	15
	SBL	4:00 pm to 4:15 pm	4	0	0	4
	SBT	4:00 pm to 4:15 pm	58	12	0	70
	SBR	4:00 pm to 4:15 pm	18	2	0	20
	WBR	4:15 pm to 4:30 pm	11	5	0	16
	SBL	4:15 pm to 4:30 pm	2	1	0	3
	SBT	4:15 pm to 4:30 pm	56	15	2	73
	SBR	4:15 pm to 4:30 pm	16	2	1	19
	WBR	4:30 pm to 4:45 pm	10	0	0	10
	SBL	4:30 pm to 4:45 pm	2	1	0	3
	SBT	4:30 pm to 4:45 pm	62	12	1	75
	SBR	4:30 pm to 4:45 pm	15	1	0	16
	WBR	4:45 pm to 5:00 pm	14	4	0	18
	SBL	4:45 pm to 5:00 pm	4	0	0	4
	SBT	4:45 pm to 5:00 pm	56	6	0	62
	SBR	4:45 pm to 5:00 pm	15	1	0	16
	WBR	5:00 pm to 5:15 pm	15	1	0	16
	SBL	5:00 pm to 5:15 pm	8	1	0	9
	SBT	5:00 pm to 5:15 pm	74	13	0	87
	SBR	5:00 pm to 5:15 pm	22	0	0	22
	WBR	5:15 pm to 5:30 pm	10	2	0	12
	SBL	5:15 pm to 5:30 pm	5	1	0	6
	SBT	5:15 pm to 5:30 pm	57	5	1	63
	SBR	5:15 pm to 5:30 pm	17	1	0	18
	WBR	5:30 pm to 5:45 pm	11	2	0	13
	SBL	5:30 pm to 5:45 pm	3	3	0	6
	SBT	5:30 pm to 5:45 pm	48	12	0	60
	SBR	5:30 pm to 5:45 pm	14	4	0	18
	WBR	5:45 pm to 6:00 pm	10	0	0	10
	SBL	5:45 pm to 6:00 pm	3	0	0	3
	SBT	5:45 pm to 6:00 pm	50	8	2	60
	SBR	5:45 pm to 6:00 pm	17	1	0	18

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: KV	EBR	8:00 am to 8:15 am	19	8	0	27
	NBL	8:00 am to 8:15 am	10	0	0	10
	NBT	8:00 am to 8:15 am	47	12	0	59
	NBR	8:00 am to 8:15 am	9	2	0	11
	EBR	8:15 am to 8:30 am	27	2	0	29
	NBL	8:15 am to 8:30 am	12	1	0	13
	NBT	8:15 am to 8:30 am	53	9	2	64
	NBR	8:15 am to 8:30 am	4	1	0	5
	EBR	8:30 am to 8:45 am	14	6	0	20
	NBL	8:30 am to 8:45 am	4	2	0	6
	NBT	8:30 am to 8:45 am	46	8	0	54
	NBR	8:30 am to 8:45 am	9	1	0	10
	EBR	8:45 am to 9:00 am	16	3	0	19
	NBL	8:45 am to 9:00 am	5	3	1	9
	NBT	8:45 am to 9:00 am	27	11	0	38
	NBR	8:45 am to 9:00 am	10	4	0	14
	EBR	9:00 am to 9:15 am	14	6	1	21
	NBL	9:00 am to 9:15 am	6	0	1	7
	NBT	9:00 am to 9:15 am	38	14	0	52
	NBR	9:00 am to 9:15 am	6	3	0	9
	EBR	9:15 am to 9:30 am	19	5	0	24
	NBL	9:15 am to 9:30 am	6	1	0	7
	NBT	9:15 am to 9:30 am	44	14	0	58
	NBR	9:15 am to 9:30 am	7	3	0	10
	EBR	9:30 am to 9:45 am	16	4	0	20
	NBL	9:30 am to 9:45 am	10	1	0	11
	NBT	9:30 am to 9:45 am	45	13	2	60
	NBR	9:30 am to 9:45 am	7	1	0	8
	EBR	9:45 am to 10:00 am	11	3	0	14
	NBL	9:45 am to 10:00 am	6	0	0	6
	NBT	9:45 am to 10:00 am	30	14	1	45
	NBR	9:45 am to 10:00 am	10	2	0	12

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: KV	EBR	4:00 pm to 4:15 pm	45	3	0	48
	NBL	4:00 pm to 4:15 pm	8	2	0	10
	NBT	4:00 pm to 4:15 pm	64	15	1	80
	NBR	4:00 pm to 4:15 pm	21	2	0	23
	EBR	4:15 pm to 4:30 pm	25	1	0	26
	NBL	4:15 pm to 4:30 pm	7	0	0	7
	NBT	4:15 pm to 4:30 pm	50	8	0	58
	NBR	4:15 pm to 4:30 pm	13	2	0	15
	EBR	4:30 pm to 4:45 pm	30	74	1	105
	NBL	4:30 pm to 4:45 pm	8	2	0	10
	NBT	4:30 pm to 4:45 pm	58	11	0	69
	NBR	4:30 pm to 4:45 pm	10	2	0	12
	EBR	4:45 pm to 5:00 pm	27	2	0	29
	NBL	4:45 pm to 5:00 pm	6	2	0	8
	NBT	4:45 pm to 5:00 pm	56	7	2	65
	NBR	4:45 pm to 5:00 pm	14	1	0	15
	EBR	5:00 pm to 5:15 pm	27	3	0	30
	NBL	5:00 pm to 5:15 pm	8	1	0	9
	NBT	5:00 pm to 5:15 pm	46	14	0	60
	NBR	5:00 pm to 5:15 pm	12	2	0	14
	EBR	5:15 pm to 5:30 pm	27	5	0	32
	NBL	5:15 pm to 5:30 pm	16	1	0	17
	NBT	5:15 pm to 5:30 pm	65	8	1	74
	NBR	5:15 pm to 5:30 pm	12	1	0	13
	EBR	5:30 pm to 5:45 pm	22	5	0	27
	NBL	5:30 pm to 5:45 pm	7	1	0	8
	NBT	5:30 pm to 5:45 pm	39	13	1	53
	NBR	5:30 pm to 5:45 pm	15	2	0	17
	EBR	5:45 pm to 6:00 pm	31	5	1	37
	NBL	5:45 pm to 6:00 pm	14	1	1	16
	NBT	5:45 pm to 6:00 pm	58	17	0	75
	NBR	5:45 pm to 6:00 pm	5	4	0	9

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2-UB	SBT	8:00 am to 8:15 am	44	11	3	58
	SBU	8:00 am to 8:15 am	8	7	0	15
	NBT	8:00 am to 8:15 am	59	6	0	65
	SBT	8:15 am to 8:30 am	44	8	2	54
	SBU	8:15 am to 8:30 am	14	1	0	15
	NBT	8:15 am to 8:30 am	59	8	2	69
	SBT	8:30 am to 8:45 am	41	13	1	55
	SBU	8:30 am to 8:45 am	10	2	0	12
	NBT	8:30 am to 8:45 am	49	9	1	59
	SBT	8:45 am to 9:00 am	40	10	2	52
	SBU	8:45 am to 9:00 am	6	1	0	7
	NBT	8:45 am to 9:00 am	41	15	1	57
	SBT	9:00 am to 9:15 am	37	15	0	52
	SBU	9:00 am to 9:15 am	5	4	1	10
	NBT	9:00 am to 9:15 am	45	12	2	59
	SBT	9:15 am to 9:30 am	41	13	2	56
	SBU	9:15 am to 9:30 am	8	1	0	9
	NBT	9:15 am to 9:30 am	49	15	0	64
	SBT	9:30 am to 9:45 am	43	12	1	56
	SBU	9:30 am to 9:45 am	8	0	0	8
	NBT	9:30 am to 9:45 am	56	15	2	73
	SBT	9:45 am to 10:00 am	36	16	1	53
	SBU	9:45 am to 10:00 am	6	1	0	7
	NBT	9:45 am to 10:00 am	42	14	2	58
	SBT	4:00 pm to 4:15 pm	75	10	0	85
	SBU	4:00 pm to 4:15 pm	27	2	0	29
	NBT	4:00 pm to 4:15 pm	67	14	0	81
	SBT	4:15 pm to 4:30 pm	68	15	1	84
	SBU	4:15 pm to 4:30 pm	14	1	0	15
	NBT	4:15 pm to 4:30 pm	56	9	0	65
	SBT	4:30 pm to 4:45 pm	76	13	0	89
	SBU	4:30 pm to 4:45 pm	20	4	0	24
	NBT	4:30 pm to 4:45 pm	59	11	1	71
	SBT	4:45 pm to 5:00 pm	67	8	0	75
	SBU	4:45 pm to 5:00 pm	17	0	0	17
	NBT	4:45 pm to 5:00 pm	61	9	1	71
	SBT	5:00 pm to 5:15 pm	84	12	0	96
	SBU	5:00 pm to 5:15 pm	16	2	0	18
	NBT	5:00 pm to 5:15 pm	51	12	1	64
	SBT	5:15 pm to 5:30 pm	68	9	0	77
	SBU	5:15 pm to 5:30 pm	16	1	0	17
	NBT	5:15 pm to 5:30 pm	79	11	1	91
	SBT	5:30 pm to 5:45 pm	61	10	0	71
	SBU	5:30 pm to 5:45 pm	10	6	0	16
	NBT	5:30 pm to 5:45 pm	56	10	1	67
	SBT	5:45 pm to 6:00 pm	71	6	2	79
	SBU	5:45 pm to 6:00 pm	17	4	0	21
	NBT	5:45 pm to 6:00 pm	60	17	1	78

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: TZ	SBT	8:00 am to 8:15 am	50	12	1	63
	SBU	8:00 am to 8:15 am	12	0	0	12
	NBT	8:00 am to 8:15 am	36	12	3	51
	SBT	8:15 am to 8:30 am	59	9	2	70
	SBU	8:15 am to 8:30 am	9	1	0	10
	NBT	8:15 am to 8:30 am	41	11	2	54
	SBT	8:30 am to 8:45 am	48	8	1	57
	SBU	8:30 am to 8:45 am	8	6	0	14
	NBT	8:30 am to 8:45 am	35	15	0	50
	SBT	8:45 am to 9:00 am	31	11	0	42
	SBU	8:45 am to 9:00 am	8	3	1	12
	NBT	8:45 am to 9:00 am	36	12	3	51
	SBT	9:00 am to 9:15 am	42	13	0	55
	SBU	9:00 am to 9:15 am	8	3	0	11
	NBT	9:00 am to 9:15 am	38	12	1	51
	SBT	9:15 am to 9:30 am	46	13	0	59
	SBU	9:15 am to 9:30 am	13	5	0	18
	NBT	9:15 am to 9:30 am	36	6	1	43
	SBT	9:30 am to 9:45 am	45	14	2	61
	SBU	9:30 am to 9:45 am	10	6	0	16
	NBT	9:30 am to 9:45 am	44	9	0	53
	SBT	9:45 am to 10:00 am	37	16	1	54
	SBU	9:45 am to 10:00 am	4	2	0	6
	NBT	9:45 am to 10:00 am	38	15	3	56
	SBT	4:00 pm to 4:15 pm	66	16	0	82
	SBU	4:00 pm to 4:15 pm	6	4	0	10
	NBT	4:00 pm to 4:15 pm	69	9	0	78
	SBT	4:15 pm to 4:30 pm	56	8	0	64
	SBU	4:15 pm to 4:30 pm	6	6	0	12
	NBT	4:15 pm to 4:30 pm	72	12	3	87
	SBT	4:30 pm to 4:45 pm	64	13	0	77
	SBU	4:30 pm to 4:45 pm	5	0	0	5
	NBT	4:30 pm to 4:45 pm	74	13	1	88
	SBT	4:45 pm to 5:00 pm	62	8	1	71
	SBU	4:45 pm to 5:00 pm	10	3	0	13
	NBT	4:45 pm to 5:00 pm	64	5	0	69
	SBT	5:00 pm to 5:15 pm	50	14	0	64
	SBU	5:00 pm to 5:15 pm	10	2	0	12
	NBT	5:00 pm to 5:15 pm	95	12	0	107
	SBT	5:15 pm to 5:30 pm	72	9	0	81
	SBU	5:15 pm to 5:30 pm	8	1	0	9
	NBT	5:15 pm to 5:30 pm	70	8	0	78
	SBT	5:30 pm to 5:45 pm	39	15	2	56
	SBU	5:30 pm to 5:45 pm	8	1	0	9
	NBT	5:30 pm to 5:45 pm	59	14	0	73
	SBT	5:45 pm to 6:00 pm	61	18	0	79
	SBU	5:45 pm to 6:00 pm	8	0	0	8
	NBT	5:45 pm to 6:00 pm	62	10	1	73

A.1.6 RCUT at North Bend

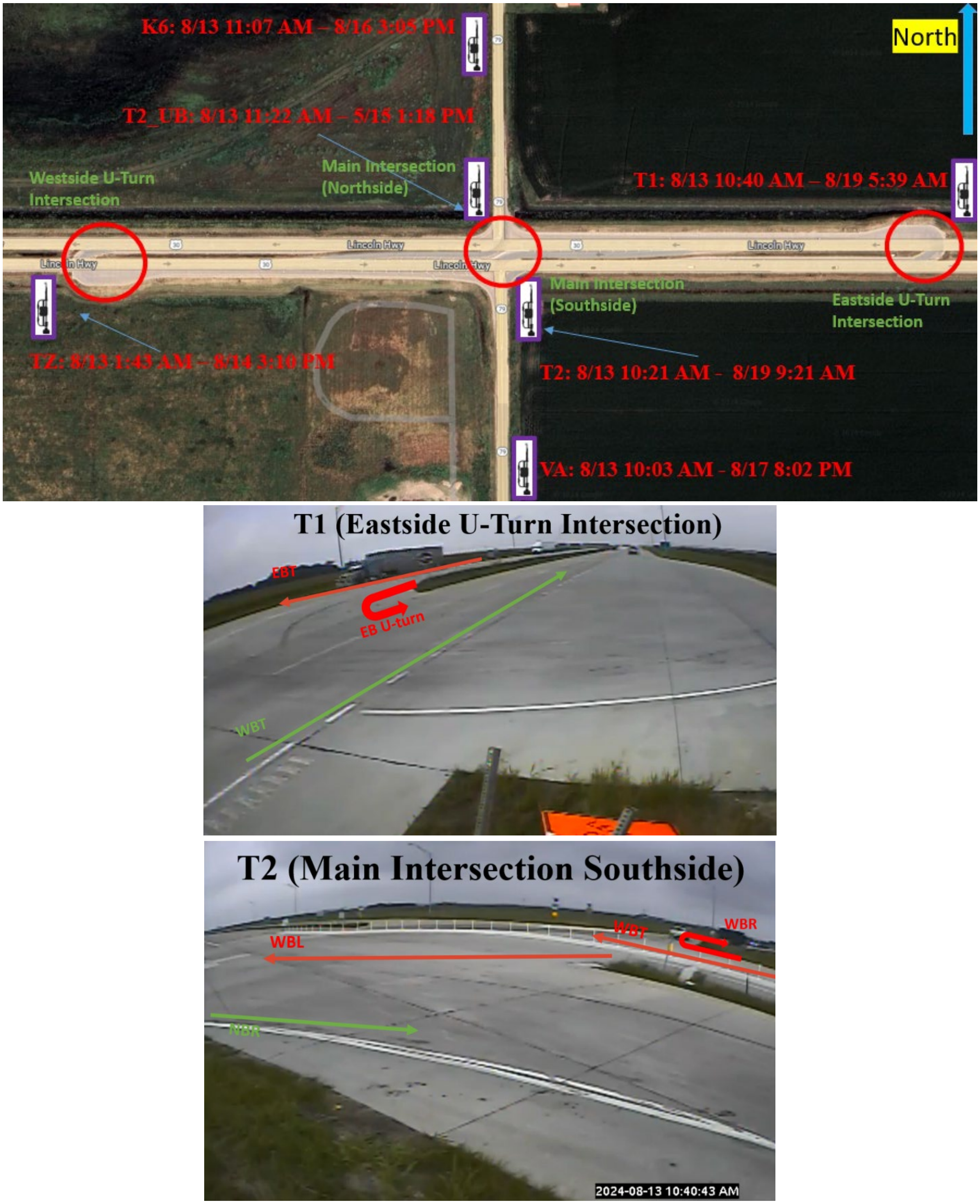


Figure A.6 Data collection device locations and corresponding traffic movements at the North Bend site

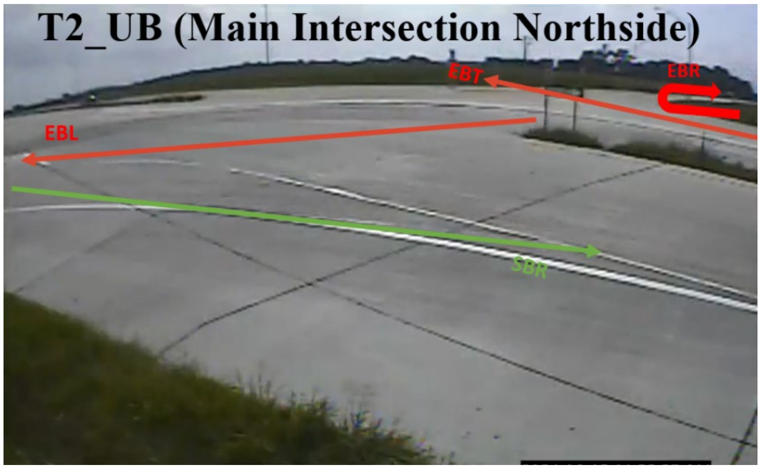


Figure A.6 cont. Data collection device locations and corresponding traffic movements at the North Bend site

Table A.6 Traffic volume counts at the North Bend site

Date: 8/14/2024 (4pm-6pm), 8/15/2024 (8am-10am)

Recording Device	Movements	Time	Vehicle Classification				Movements	Time	Vehicle Classification			
			PC	TT	T	Total			PC	TT	T	Total
Miovision: T2	NBR	4:00 pm to 4:15 pm	26	4	0	30	NBR	8:00 am to 8:15 am	33	1	1	35
	WBL	4:00 pm to 4:15 pm	18	1	0	19	WBL	8:00 am to 8:15 am	1	2	0	3
	WBT	4:00 pm to 4:15 pm	52	14	1	67	WBT	8:00 am to 8:15 am	41	8	2	51
	WBR	4:00 pm to 4:15 pm	8	4	0	12	WBR	8:00 am to 8:15 am	7	0	0	7
	NBR	4:15 pm to 4:30 pm	21	2	2	25	NBR	8:15 am to 8:30 am	24	1	2	27
	WBL	4:15 pm to 4:30 pm	19	0	1	20	WBL	8:15 am to 8:30 am	10	2	0	12
	WBT	4:15 pm to 4:30 pm	52	12	1	65	WBT	8:15 am to 8:30 am	32	7	1	40
	WBR	4:15 pm to 4:30 pm	10	1	0	11	WBR	8:15 am to 8:30 am	8	0	1	9
	NBR	4:30 pm to 4:45 pm	23	2	1	26	NBR	8:30 am to 8:45 am	12	3	0	15
	WBL	4:30 pm to 4:45 pm	39	2	1	42	WBL	8:30 am to 8:45 am	4	0	0	4
	WBT	4:30 pm to 4:45 pm	53	8	1	62	WBT	8:30 am to 8:45 am	31	15	3	49
	WBR	4:30 pm to 4:45 pm	6	0	1	7	WBR	8:30 am to 8:45 am	4	0	0	4
	NBR	4:45 pm to 5:00 pm	29	7	2	38	NBR	8:45 am to 9:00 am	12	4	0	16
	WBL	4:45 pm to 5:00 pm	29	2	0	31	WBL	8:45 am to 9:00 am	4	4	1	9
	WBT	4:45 pm to 5:00 pm	51	9	0	60	WBT	8:45 am to 9:00 am	34	12	1	47
	WBR	4:45 pm to 5:00 pm	11	1	0	12	WBR	8:45 am to 9:00 am	5	1	1	7
	NBR	5:00 pm to 5:15 pm	41	4	0	45	NBR	9:00 am to 9:15 am	22	3	0	25
	WBL	5:00 pm to 5:15 pm	21	1	0	22	WBL	9:00 am to 9:15 am	12	0	2	14
	WBT	5:00 pm to 5:15 pm	45	13	0	58	WBT	9:00 am to 9:15 am	27	20	3	50
	WBR	5:00 pm to 5:15 pm	16	1	0	17	WBR	9:00 am to 9:15 am	9	3	1	13
	NBR	5:15 pm to 5:30 pm	27	0	0	27	NBR	9:15 am to 9:30 am	23	2	1	26
	WBL	5:15 pm to 5:30 pm	22	0	0	22	WBL	9:15 am to 9:30 am	12	3	1	16
	WBT	5:15 pm to 5:30 pm	51	3	0	54	WBT	9:15 am to 9:30 am	35	10	2	47
	WBR	5:15 pm to 5:30 pm	13	0	0	13	WBR	9:15 am to 9:30 am	8	0	0	8
	NBR	5:30 pm to 5:45 pm	22	4	0	26	NBR	9:30 am to 9:45 am	20	4	1	25
	WBL	5:30 pm to 5:45 pm	27	2	0	29	WBL	9:30 am to 9:45 am	9	0	0	9
	WBT	5:30 pm to 5:45 pm	56	5	0	61	WBT	9:30 am to 9:45 am	37	15	1	53
	WBR	5:30 pm to 5:45 pm	8	0	0	8	WBR	9:30 am to 9:45 am	3	1	0	4
	NBR	5:45 pm to 6:00 pm	52	3	0	55	NBR	9:45 am to 10:00 am	40	3	0	43
	WBL	5:45 pm to 6:00 pm	7	4	0	11	WBL	9:45 am to 10:00 am	27	1	0	28
	WBT	5:45 pm to 6:00 pm	21	16	2	39	WBT	9:45 am to 10:00 am	53	5	0	58
	WBR	5:45 pm to 6:00 pm	12	2	0	14	WBR	9:45 am to 10:00 am	21	0	0	21

Recording Device	Movements	Time	Vehicle Classification				Movements	Time	Vehicle Classification			
			PC	TT	T	Total			PC	TT	T	Total
Miovision: T2-UB	SBR	4:00 pm to 4:15 pm	9	1	1	11	SBR	8:00 am to 8:15 am	6	4	0	10
	EBL	4:00 pm to 4:15 pm	2	0	0	2	EBL	8:00 am to 8:15 am	0	1	0	1
	EBT	4:00 pm to 4:15 pm	46	7	3	56	EBT	8:00 am to 8:15 am	43	11	2	56
	EBR	4:00 pm to 4:15 pm	16	0	0	16	EBR	8:00 am to 8:15 am	7	1	0	8
	SBR	4:15 pm to 4:30 pm	10	0	0	10	SBR	8:15 am to 8:30 am	6	4	0	10
	EBL	4:15 pm to 4:30 pm	2	0	0	2	EBL	8:15 am to 8:30 am	0	1	0	1
	EBT	4:15 pm to 4:30 pm	47	12	0	59	EBT	8:15 am to 8:30 am	35	16	2	53
	EBR	4:15 pm to 4:30 pm	18	0	0	18	EBR	8:15 am to 8:30 am	15	3	0	18
	SBR	4:30 pm to 4:45 pm	9	0	0	9	SBR	8:30 am to 8:45 am	9	8	0	17
	EBL	4:30 pm to 4:45 pm	5	1	0	6	EBL	8:30 am to 8:45 am	7	1	0	8
	EBT	4:30 pm to 4:45 pm	46	11	2	59	EBT	8:30 am to 8:45 am	33	11	1	45
	EBR	4:30 pm to 4:45 pm	24	1	1	26	EBR	8:30 am to 8:45 am	15	2	0	17
	SBR	4:45 pm to 5:00 pm	36	1	0	37	SBR	8:45 am to 9:00 am	9	1	0	10
	EBL	4:45 pm to 5:00 pm	3	2	0	5	EBL	8:45 am to 9:00 am	3	2	0	5
	EBT	4:45 pm to 5:00 pm	58	7	1	66	EBT	8:45 am to 9:00 am	29	18	1	48
	EBR	4:45 pm to 5:00 pm	37	1	0	38	EBR	8:45 am to 9:00 am	13	3	0	16
	SBR	5:00 pm to 5:15 pm	10	1	0	11	SBR	9:00 am to 9:15 am	8	6	0	14
	EBL	5:00 pm to 5:15 pm	2	1	0	3	EBL	9:00 am to 9:15 am	3	5	0	8
	EBT	5:00 pm to 5:15 pm	37	8	0	45	EBT	9:00 am to 9:15 am	44	6	1	51
	EBR	5:00 pm to 5:15 pm	15	0	0	15	EBR	9:00 am to 9:15 am	12	2	0	14
	SBR	5:15 pm to 5:30 pm	9	3	0	12	SBR	9:15 am to 9:30 am	11	3	1	15
	EBL	5:15 pm to 5:30 pm	4	1	0	5	EBL	9:15 am to 9:30 am	1	3	1	5
	EBT	5:15 pm to 5:30 pm	34	9	0	43	EBT	9:15 am to 9:30 am	24	19	3	46
	EBR	5:15 pm to 5:30 pm	25	0	0	25	EBR	9:15 am to 9:30 am	13	7	0	20
	SBR	5:30 pm to 5:45 pm	10	2	1	13	SBR	9:30 am to 9:45 am	8	4	0	12
	EBL	5:30 pm to 5:45 pm	3	0	0	3	EBL	9:30 am to 9:45 am	1	1	1	3
	EBT	5:30 pm to 5:45 pm	36	2	2	40	EBT	9:30 am to 9:45 am	41	14	2	57
	EBR	5:30 pm to 5:45 pm	21	0	0	21	EBR	9:30 am to 9:45 am	9	3	0	12
	SBR	5:45 pm to 6:00 pm	18	0	0	18	SBR	9:45 am to 10:00 am	4	1	0	5
	EBL	5:45 pm to 6:00 pm	4	0	0	4	EBL	9:45 am to 10:00 am	0	2	0	2
	EBT	5:45 pm to 6:00 pm	38	7	0	45	EBT	9:45 am to 10:00 am	49	15	2	66
	EBR	5:45 pm to 6:00 pm	27	0	0	27	EBR	9:45 am to 10:00 am	10	2	0	12

Recording Device	Movements	Time	Vehicle Classification				Movements	Time	Vehicle Classification			
			PC	TT	T	Total			PC	TT	T	Total
Miovision: T1	EBT	4:00 pm to 4:15 pm	60	11	1	72	EBT	8:00 am to 8:15 am	72	10	0	82
	EBU	4:00 pm to 4:15 pm	13	3	1	17	EBU	8:00 am to 8:15 am	25	4	0	29
	WBT	4:00 pm to 4:15 pm	62	15	1	78	WBT	8:00 am to 8:15 am	38	18	2	58
	EBT	4:15 pm to 4:30 pm	51	17	1	69	EBT	8:15 am to 8:30 am	57	13	1	71
	EBU	4:15 pm to 4:30 pm	14	0	0	14	EBU	8:15 am to 8:30 am	11	0	1	12
	WBT	4:15 pm to 4:30 pm	71	9	3	83	WBT	8:15 am to 8:30 am	38	9	3	50
	EBT	4:30 pm to 4:45 pm	57	8	2	67	EBT	8:30 am to 8:45 am	55	16	0	71
	EBU	4:30 pm to 4:45 pm	14	2	0	16	EBU	8:30 am to 8:45 am	10	0	0	10
	WBT	4:30 pm to 4:45 pm	90	6	1	97	WBT	8:30 am to 8:45 am	40	8	0	48
	EBT	4:45 pm to 5:00 pm	72	11	2	85	EBT	8:45 am to 9:00 am	37	18	0	55
	EBU	4:45 pm to 5:00 pm	17	6	0	23	EBU	8:45 am to 9:00 am	6	4	0	10
	WBT	4:45 pm to 5:00 pm	78	9	0	87	WBT	8:45 am to 9:00 am	39	14	3	56
	EBT	5:00 pm to 5:15 pm	55	8	1	64	EBT	9:00 am to 9:15 am	51	7	0	58
	EBU	5:00 pm to 5:15 pm	25	3	0	28	EBU	9:00 am to 9:15 am	5	3	0	8
	WBT	5:00 pm to 5:15 pm	59	9	0	68	WBT	9:00 am to 9:15 am	38	11	3	52
	EBT	5:15 pm to 5:30 pm	48	7	0	55	EBT	9:15 am to 9:30 am	36	20	2	58
	EBU	5:15 pm to 5:30 pm	22	1	0	23	EBU	9:15 am to 9:30 am	9	2	0	11
	WBT	5:15 pm to 5:30 pm	68	3	0	71	WBT	9:15 am to 9:30 am	40	17	5	62
	EBT	5:30 pm to 5:45 pm	47	6	0	53	EBT	9:30 am to 9:45 am	55	16	0	71
	EBU	5:30 pm to 5:45 pm	15	1	0	16	EBU	9:30 am to 9:45 am	9	2	1	12
	WBT	5:30 pm to 5:45 pm	77	4	0	81	WBT	9:30 am to 9:45 am	45	11	2	58
	EBT	5:45 pm to 6:00 pm	51	10	0	61	EBT	9:45 am to 10:00 am	59	14	1	74
	EBU	5:45 pm to 6:00 pm	21	1	0	22	EBU	9:45 am to 10:00 am	10	2	0	12
	WBT	5:45 pm to 6:00 pm	76	5	0	81	WBT	9:45 am to 10:00 am	45	14	1	60

A.1.7 RCUT at Murray

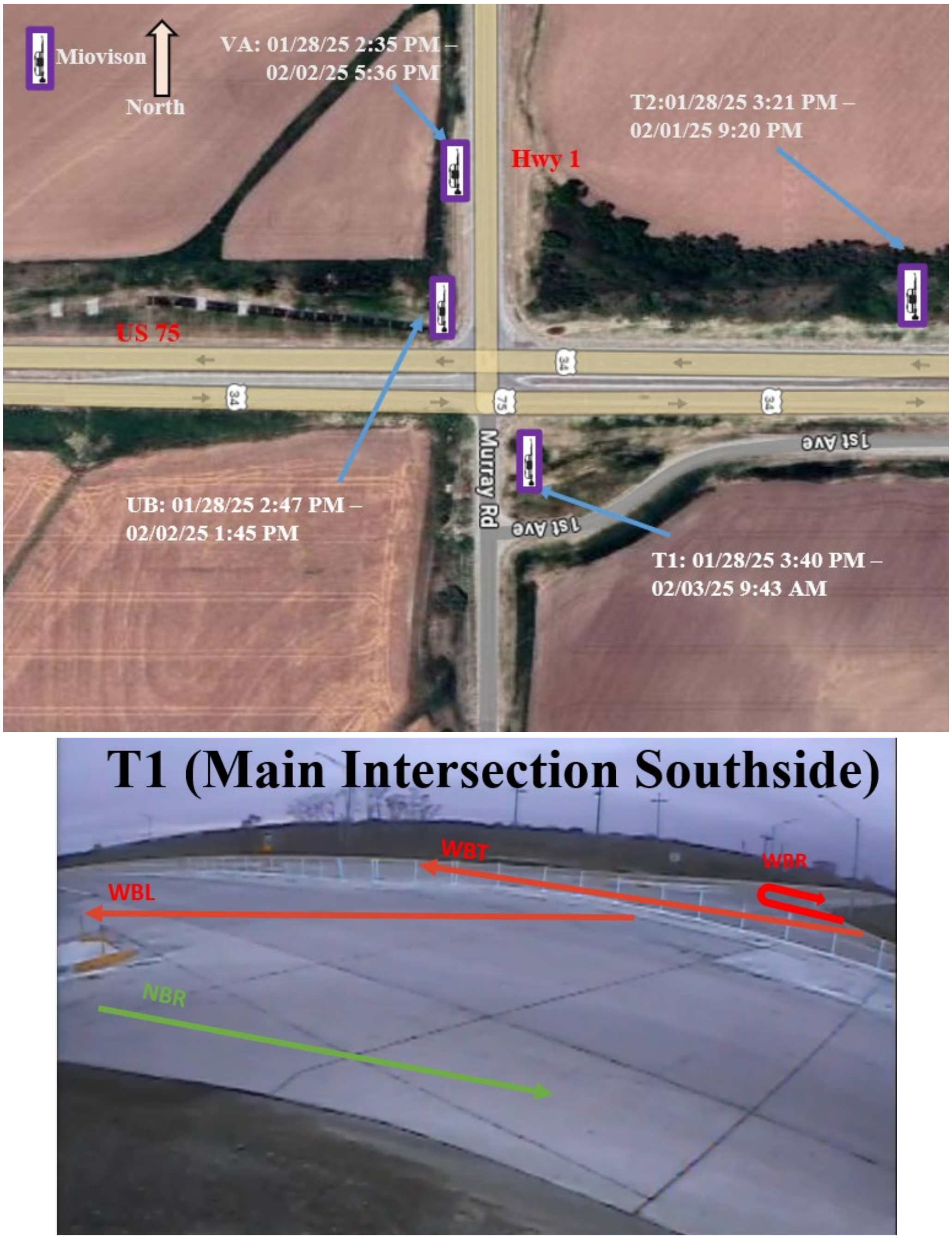


Figure A.7 Data collection device locations and corresponding traffic movements at the Murray site

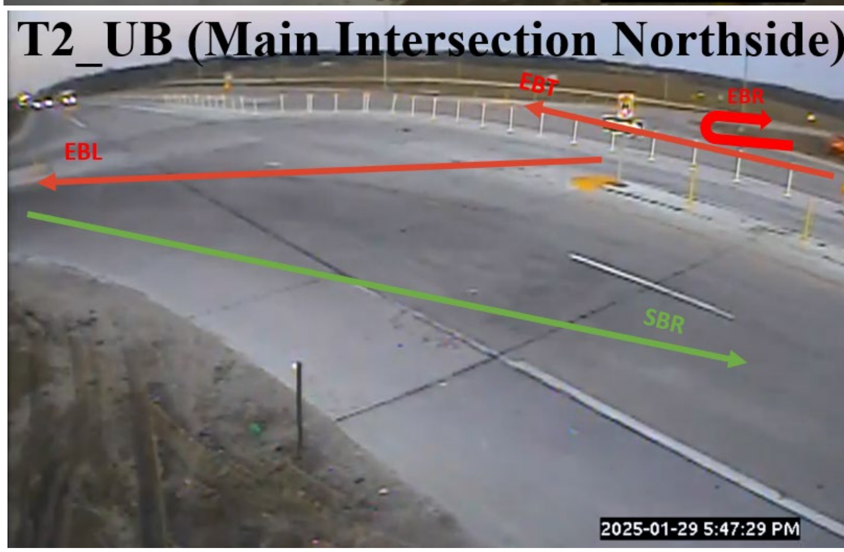


Figure A.7 cont. Data collection device locations and corresponding traffic movements at the Murray site

Table A.7 Traffic volume counts at the Murray site

Date: 1/29/2025

Recording device	Movements	Time	Vehicle Classificaion			
			PC	TT	T	Total
Miovision: T1	WBR	4:00 PM TO 4:15 PM	29	4	1	34
	WBT	4:00 PM TO 4:15 PM	73	6	0	69
	WBL	4:00 PM TO 4:15 PM	35	0	0	35
	NBR	4:00 PM TO 4:15 PM	20	1	1	22
	WBR	4:15 PM TO 4:30 PM	36	0	3	39
	WBT	4:15 PM TO 4:30 PM	59	5	0	64
	WBL	4:15 PM TO 4:30 PM	26	0	1	27
	NBR	4:15 PM TO 4:30 PM	18	0	2	20
	WBR	4:30 PM TO 4:45 PM	26	1	2	29
	WBT	4:30 PM TO 4:45 PM	68	3	0	71
	WBL	4:30 PM TO 4:45 PM	24	0	1	25
	NBR	4:30 PM TO 4:45 PM	23	2	2	27
	WBR	4:45 PM TO 5:00 PM	36	0	2	38
	WBT	4:45 PM TO 5:00 PM	44	6	1	51
	WBL	4:45 PM TO 5:00 PM	28	0	0	28
	NBR	4:45 PM TO 5:00 PM	18	0	1	19
	WBR	5:00 PM TO 5:15 PM	48	1	1	50
	WBT	5:00 PM TO 5:15 PM	47	3	0	50
	WBL	5:00 PM TO 5:15 PM	18	0	0	18
	NBR	5:00 PM TO 5:15 PM	18	0	1	19
	WBR	5:15 PM TO 5:30 PM	42	2	0	44
	WBT	5:15 PM TO 5:30 PM	57	1	1	59
	WBL	5:15 PM TO 5:30 PM	24	0	0	24
	NBR	5:15 PM TO 5:30 PM	23	0	0	23
	WBR	5:30 PM TO 5:45 PM	35	1	0	36
	WBT	5:30 PM TO 5:45 PM	41	2	0	43
	WBL	5:30 PM TO 5:45 PM	34	0	0	34
	NBR	5:30 PM TO 5:45 PM	27	0	0	27
	WBR	5:45 PM TO 6:00 PM	30	0	0	30
	WBT	5:45 PM TO 6:00 PM	38	2	2	42
	WBL	5:45 PM TO 6:00 PM	29	0	0	29
	NBR	5:45 PM TO 6:00 PM	19	0	0	19

Recording device	Movements	Time	Vehicle Classificaion			
			PC	TT	T	Total
Miovision: T2	EBT	4:00 PM TO 4:15 PM	91	9	4	104
	EB U-turn	4:00 PM TO 4:15 PM	4	1	0	5
	WBT	4:00 PM TO 4:15 PM	120	9	1	130
	EBT	4:15 PM TO 4:30 PM	81	3	5	89
	EB U-turn	4:15 PM TO 4:30 PM	11	0	0	11
	WBT	4:15 PM TO 4:30 PM	101	4	4	109
	EBT	4:30 PM TO 4:45 PM	93	3	3	99
	EB U-turn	4:30 PM TO 4:45 PM	8	0	2	10
	WBT	4:30 PM TO 4:45 PM	118	3	2	123
	EBT	4:45 PM TO 5:00 PM	76	5	2	83
	EB U-turn	4:45 PM TO 5:00 PM	5	0	1	6
	WBT	4:45 PM TO 5:00 PM	90	4	3	97
	EBT	5:00 PM TO 5:15 PM	84	1	0	85
	EB U-turn	5:00 PM TO 5:15 PM	7	0	1	8
	WBT	5:00 PM TO 5:15 PM	119	5	1	125
	EBT	5:15 PM TO 5:30 PM	82	1	1	84
	EB U-turn	5:15 PM TO 5:30 PM	5	0	0	5
	WBT	5:15 PM TO 5:30 PM	105	2	0	107
	EBT	5:30 PM TO 5:45 PM	66	1	0	67
	EB U-turn	5:30 PM TO 5:45 PM	8	0	0	8
	WBT	5:30 PM TO 5:45 PM	113	3	0	116
	EBT	5:45 PM TO 6:00 PM	52	1	1	54
	EB U-turn	5:45 PM TO 6:00 PM	2	0	0	2
	WBT	5:45 PM TO 6:00 PM	86	2	2	90

Miovision: T2_UB	EBR	4:00 PM TO 4:15 PM	29	0	1	30
	EBT	4:00 PM TO 4:15 PM	74	8	3	85
	EBL	4:00 PM TO 4:15 PM	2	0	0	2
	SBR	4:00 PM TO 4:15 PM	84	0	2	86
	EBR	4:15 PM TO 4:30 PM	11	0	1	12
	EBT	4:15 PM TO 4:30 PM	72	3	2	77
	EBL	4:15 PM TO 4:30 PM	1	0	0	1
	SBR	4:15 PM TO 4:30 PM	37	1	2	39
	EBR	4:30 PM TO 4:45 PM	14	0	1	15
	EBT	4:30 PM TO 4:45 PM	73	3	3	79
	EBL	4:30 PM TO 4:45 PM	2	0	0	2
	SBR	4:30 PM TO 4:45 PM	49	0	2	51
	EBR	4:45 PM TO 5:00 PM	13	0	0	13
	EBT	4:45 PM TO 5:00 PM	63	5	3	71
	EBL	4:45 PM TO 5:00 PM	3	0	0	3
	SBR	4:45 PM TO 5:00 PM	24	0	0	24
	EBR	5:00 PM TO 5:15 PM	10	0	0	10
	EBT	5:00 PM TO 5:15 PM	63	1	0	64
	EBL	5:00 PM TO 5:15 PM	1	0	0	1
	SBR	5:00 PM TO 5:15 PM	32	0	1	33
	EBR	5:15 PM TO 5:30 PM	14	0	0	14
	EBT	5:15 PM TO 5:30 PM	67	1	1	69
	EBL	5:15 PM TO 5:30 PM	0	0	0	0
	SBR	5:15 PM TO 5:30 PM	23	0	0	23
	EBR	5:30 PM TO 5:45 PM	16	0	0	16
	EBT	5:30 PM TO 5:45 PM	47	1	3	51
	EBL	5:30 PM TO 5:45 PM	2	0	0	2
	SBR	5:30 PM TO 5:45 PM	18	0	0	18
	EBR	5:45 PM TO 6:00 PM	6	0	0	6
	EBT	5:45 PM TO 6:00 PM	34	1	1	36
	EBL	5:45 PM TO 6:00 PM	0	0	0	0
	SBR	5:45 PM TO 6:00 PM	11	0	0	11

Recording device	Movements	Time	Vehicle Classificaion			
			PC	TT	T	Total
Miovision: T1	WBR	11:00 AM - 11:15 AM	15	2	0	17
	WBT	11:00 AM - 11:15 AM	29	6	1	36
	WBL	11:00 AM - 11:15 AM	4	0	0	4
	NBR	11:00 AM - 11:15 AM	19	0	0	19
	WBR	11:15 AM - 11:30 AM	12	4	0	16
	WBT	11:15 AM - 11:30 AM	26	2	1	29
	WBL	11:15 AM - 11:30 AM	6	0	0	6
	NBR	11:15 AM - 11:30 AM	15	1	0	16
	WBR	11:30 AM - 11:45 AM	14	3	1	18
	WBT	11:30 AM - 11:45 AM	28	5	1	34
	WBL	11:30 AM - 11:45 AM	13	0	0	13
	NBR	11:30 AM - 11:45 AM	13	0	0	13
	WBR	11:45 AM - 12:00 PM	14	3	1	18
	WBT	11:45 AM - 12:00 PM	22	6	0	28
	WBL	11:45 AM - 12:00 PM	9	0	0	9
	NBR	11:45 AM - 12:00 PM	9	0	0	9
	WBR	12:00 PM - 12:15 PM	10	0	1	11
	WBT	12:00 PM - 12:15 PM	27	4	0	31
	WBL	12:00 PM - 12:15 PM	10	0	1	11
	NBR	12:00 PM - 12:15 PM	15	0	1	16
	WBR	12:15 PM - 12:30 PM	12	1	0	13
	WBT	12:15 PM - 12:30 PM	23	7	4	34
	WBL	12:15 PM - 12:30 PM	12	0	0	12
	NBR	12:15 PM - 12:30 PM	11	0	0	11
	WBR	12:30 PM - 12:45 PM	11	0	0	11
	WBT	12:30 PM - 12:45 PM	35	6	3	44
	WBL	12:30 PM - 12:45 PM	12	0	0	12
	NBR	12:30 PM - 12:45 PM	17	0	1	18
	WBR	12:45 PM - 1:00 PM	15	1	1	17
	WBT	12:45 PM - 1:00 PM	21	8	0	29
	WBL	12:45 PM - 1:00 PM	11	0	0	11
	NBR	12:45 PM - 1:00 PM	17	0	0	17

Recording device	Movements	Time	Vehicle Classificaion			
			PC	TT	T	Total
Miovision: T2	EBT	11:00 AM - 11:15 AM	46	9	1	56
	EB U-turn	11:00 AM - 11:15 AM	4	1	0	5
	WBT	11:00 AM - 11:15 AM	43	6	1	50
	EBT	11:15 AM - 11:30 AM	53	12	0	65
	EB U-turn	11:15 AM - 11:30 AM	2	0	0	2
	WBT	11:15 AM - 11:30 AM	47	8	2	57
	EBT	11:30 AM - 11:45 AM	43	9	2	54
	EB U-turn	11:30 AM - 11:45 AM	4	0	0	4
	WBT	11:30 AM - 11:45 AM	53	6	1	60
	EBT	11:45 AM - 12:00 PM	48	3	1	52
	EB U-turn	11:45 AM - 12:00 PM	2	0	0	2
	WBT	11:45 AM - 12:00 PM	34	10	1	45
	EBT	12:00 PM - 12:15 PM	50	12	0	62
	EB U-turn	12:00 PM - 12:15 PM	4	0	0	4
	WBT	12:00 PM - 12:15 PM	43	4	2	49
	EBT	12:15 PM - 12:30 PM	55	7	2	64
	EB U-turn	12:15 PM - 12:30 PM	0	0	0	0
	WBT	12:15 PM - 12:30 PM	51	9	2	62
	EBT	12:30 PM - 12:45 PM	60	4	1	65
	EB U-turn	12:30 PM - 12:45 PM	3	0	1	4
	WBT	12:30 PM - 12:45 PM	47	5	2	54
	EBT	12:45 PM - 1:00 PM	49	9	0	58
	EB U-turn	12:45 PM - 1:00 PM	3	0	0	3
	WBT	12:45 PM - 1:00 PM	46	10	0	56

Recording device	Movements	Time	Vehicle Classificaion			
			PC	TT	T	Total
Miovision: T2_UB	EBR	11:00 AM - 11:15 AM	2	0	0	2
	EBT	11:00 AM - 11:15 AM	35	10	1	46
	EBL	11:00 AM - 11:15 AM	1	0	0	1
	SBR	11:00 AM - 11:15 AM	11	1	1	13
	EBR	11:15 AM - 11:30 AM	5	0	0	5
	EBT	11:15 AM - 11:30 AM	39	12	0	51
	EBL	11:15 AM - 11:30 AM	1	0	0	1
	SBR	11:15 AM - 11:30 AM	18	0	0	18
	EBR	11:30 AM - 11:45 AM	0	1	0	1
	EBT	11:30 AM - 11:45 AM	34	9	2	45
	EBL	11:30 AM - 11:45 AM	0	0	0	0
	SBR	11:30 AM - 11:45 AM	9	4	1	14
	EBR	11:45 AM - 12:00 PM	4	0	1	5
	EBT	11:45 AM - 12:00 PM	43	3	1	47
	EBL	11:45 AM - 12:00 PM	1	0	0	1
	SBR	11:45 AM - 12:00 PM	19	0	1	20
	EBR	12:00 PM - 12:15 PM	4	0	0	4
	EBT	12:00 PM - 12:15 PM	37	12	0	49
	EBL	12:00 PM - 12:15 PM	0	0	1	1
	SBR	12:00 PM - 12:15 PM	13	1	0	14
	EBR	12:15 PM - 12:30 PM	6	0	0	6
	EBT	12:15 PM - 12:30 PM	43	7	3	53
	EBL	12:15 PM - 12:30 PM	1	0	1	2
	SBR	12:15 PM - 12:30 PM	22	2	1	25
	EBR	12:30 PM - 12:45 PM	0	0	0	0
	EBT	12:30 PM - 12:45 PM	42	4	2	48
	EBL	12:30 PM - 12:45 PM	4	0	0	4
	SBR	12:30 PM - 12:45 PM	8	3	1	12
	EBR	12:45 PM - 1:00 PM	1	0	0	1
	EBT	12:45 PM - 1:00 PM	35	10	0	45
	EBL	12:45 PM - 1:00 PM	1	0	1	2
	SBR	12:45 PM - 1:00 PM	14	3	0	17

Recording device	Movements	Time	Vehicle Classificaion			
			PC	TT	T	Total
Miovision: T1	WBR	7:00 AM - 7:15 AM	27	0	0	27
	WBT	7:00 AM - 7:15 AM	35	3	0	38
	WBL	7:00 AM - 7:15 AM	5	0	0	5
	NBR	7:00 AM - 7:15 AM	53	0	0	53
	WBR	7:15 AM - 7:30 AM	69	0	3	72
	WBT	7:15 AM - 7:30 AM	44	2	5	51
	WBL	7:15 AM - 7:30 AM	7	0	0	7
	NBR	7:15 AM - 7:30 AM	69	0	4	73
	WBR	7:30 AM - 7:45 AM	90	0	0	90
	WBT	7:30 AM - 7:45 AM	37	3	1	41
	WBL	7:30 AM - 7:45 AM	14	0	2	16
	NBR	7:30 AM - 7:45 AM	67	0	1	68
	WBR	7:45 AM - 8:00 AM	59	0	0	59
	WBT	7:45 AM - 8:00 AM	23	2	2	27
	WBL	7:45 AM - 8:00 AM	18	0	2	20
	NBR	7:45 AM - 8:00 AM	53	0	1	54
	WBR	8:00 AM - 8:15 AM	17	3	3	23
	WBT	8:00 AM - 8:15 AM	35	4	1	40
	WBL	8:00 AM - 8:15 AM	11	0	2	13
	NBR	8:00 AM - 8:15 AM	28	0	2	30
	WBR	8:15 AM - 8:30 AM	8	0	0	8
	WBT	8:15 AM - 8:30 AM	26	7	9	42
	WBL	8:15 AM - 8:30 AM	4	0	0	4
	NBR	8:15 AM - 8:30 AM	13	0	0	13
	WBR	8:30 AM - 8:45 AM	13	1	0	14
	WBT	8:30 AM - 8:45 AM	25	4	2	31
	WBL	8:30 AM - 8:45 AM	8	0	1	9
	NBR	8:30 AM - 8:45 AM	18	0	1	19
	WBR	8:45 AM - 9:00 AM	4	4	0	8
	WBT	8:45 AM - 9:00 AM	23	7	7	37
	WBL	8:45 AM - 9:00 AM	8	0	2	10
	NBR	8:45 AM - 9:00 AM	21	0	1	22

Recording device	Movements	Time	Vehicle Classificaion			
			PC	TT	T	Total
Miovision: T2	EBT	7:00 AM - 7:15 AM	96	8	2	106
	EB U-turn	7:00 AM - 7:15 AM	12	0	0	12
	WBT	7:00 AM - 7:15 AM	55	2	1	58
	EBT	7:15 AM - 7:30 AM	109	7	3	119
	EB U-turn	7:15 AM - 7:30 AM	34	0	3	37
	WBT	7:15 AM - 7:30 AM	87	2	3	92
	EBT	7:30 AM - 7:45 AM	104	8	3	115
	EB U-turn	7:30 AM - 7:45 AM	33	0	0	33
	WBT	7:30 AM - 7:45 AM	109	3	3	115
	EBT	7:45 AM - 8:00 AM	122	8	6	136
	EB U-turn	7:45 AM - 8:00 AM	19	0	0	19
	WBT	7:45 AM - 8:00 AM	68	3	4	75
	EBT	8:00 AM - 8:15 AM	101	6	3	110
	EB U-turn	8:00 AM - 8:15 AM	4	0	0	4
	WBT	8:00 AM - 8:15 AM	54	9	4	67
	EBT	8:15 AM - 8:30 AM	41	6	1	49
	EB U-turn	8:15 AM - 8:30 AM	4	0	0	4
	WBT	8:15 AM - 8:30 AM	38	4	4	46
	EBT	8:30 AM - 8:45 AM	59	4	2	65
	EB U-turn	8:30 AM - 8:45 AM	3	0	0	3
	WBT	8:30 AM - 8:45 AM	42	5	1	48
	EBT	8:45 AM - 9:00 AM	61	7	2	70
	EB U-turn	8:45 AM - 9:00 AM	1	1	0	2
	WBT	8:45 AM - 9:00 AM	36	9	6	51

Recording device	Movements	Time	Vehicle Classificaion			
			PC	TT	T	Total
Miovision: T2_UB	EBR	7:00 AM - 7:15 AM	0	0	1	1
	EBT	7:00 AM - 7:15 AM	56	8	2	66
	EBL	7:00 AM - 7:15 AM	5	0	0	5
	SBR	7:00 AM - 7:15 AM	24	0	1	25
	EBR	7:15 AM - 7:30 AM	0	0	0	0
	EBT	7:15 AM - 7:30 AM	64	7	3	74
	EBL	7:15 AM - 7:30 AM	4	0	1	5
	SBR	7:15 AM - 7:30 AM	18	1	1	20
	EBR	7:30 AM - 7:45 AM	5	0	0	5
	EBT	7:30 AM - 7:45 AM	64	8	2	74
	EBL	7:30 AM - 7:45 AM	4	0	0	4
	SBR	7:30 AM - 7:45 AM	41	1	1	43
	EBR	7:45 AM - 8:00 AM	12	0	0	12
	EBT	7:45 AM - 8:00 AM	98	8	2	108
	EBL	7:45 AM - 8:00 AM	7	0	0	7
	SBR	7:45 AM - 8:00 AM	74	1	0	75
	EBR	8:00 AM - 8:15 AM	14	2	0	16
	EBT	8:00 AM - 8:15 AM	74	9	1	84
	EBL	8:00 AM - 8:15 AM	3	0	0	3
	SBR	8:00 AM - 8:15 AM	55	3	1	59
	EBR	8:15 AM - 8:30 AM	1	0	0	1
	EBT	8:15 AM - 8:30 AM	30	6	1	37
	EBL	8:15 AM - 8:30 AM	0	0	0	0
	SBR	8:15 AM - 8:30 AM	11	1	0	12
	EBR	8:30 AM - 8:45 AM	0	0	0	0
	EBT	8:30 AM - 8:45 AM	41	5	2	49
	EBL	8:30 AM - 8:45 AM	3	0	0	3
	SBR	8:30 AM - 8:45 AM	12	1	1	14
	EBR	8:45 AM - 9:00 AM	1	0	0	1
	EBT	8:45 AM - 9:00 AM	43	7	2	52
	EBL	8:45 AM - 9:00 AM	1	0	0	1
	SBR	8:45 AM - 9:00 AM	18	1	1	20

A.1.8 Roundabout at Norfolk

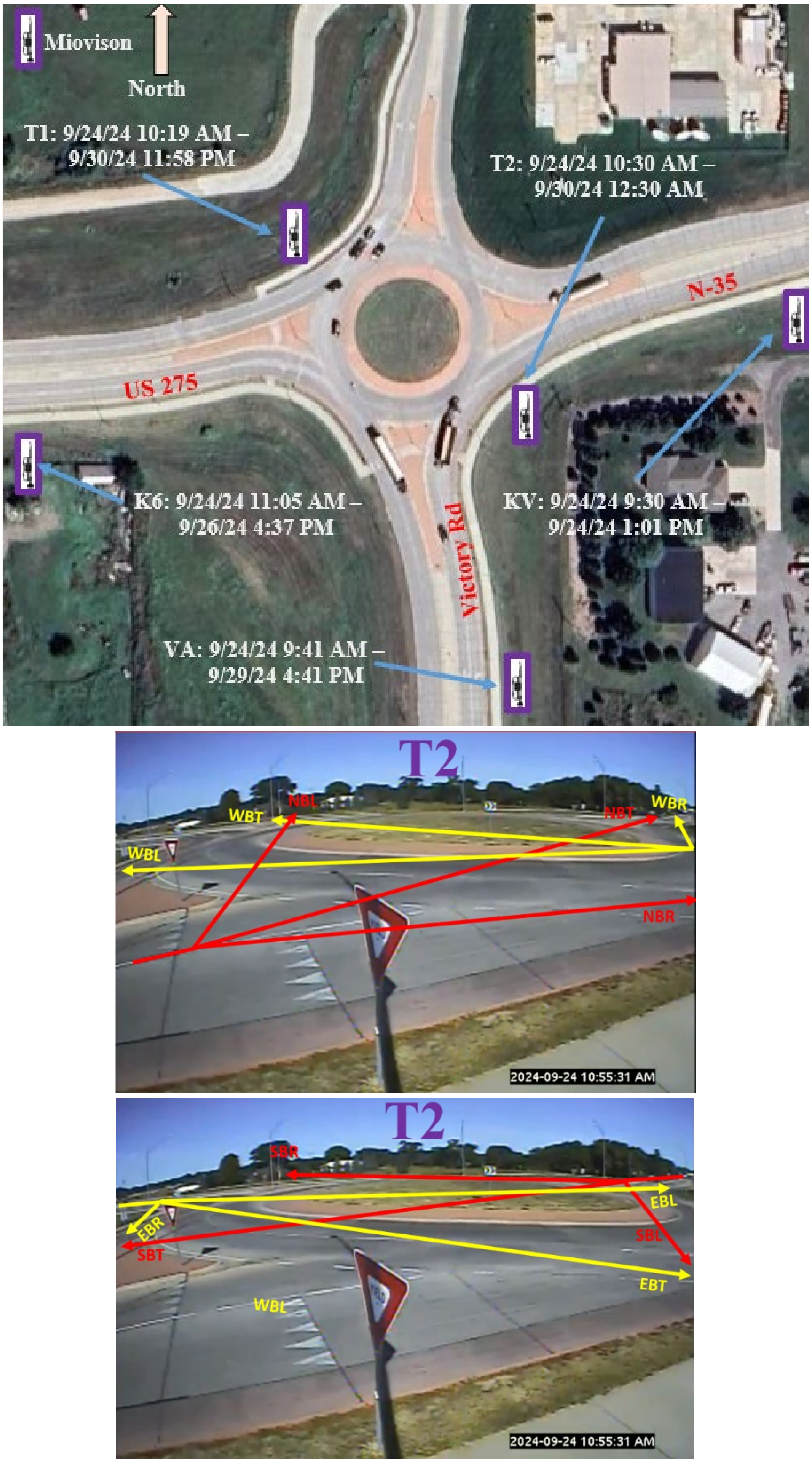


Figure A.8 Data collection device locations and corresponding traffic movements at the Norfolk site

Table A.8 Traffic volume counts at the Norfolk site

Date: 9/25/2024

Recording Device	Movements	Time	Vehicle Classification				Movements	Time	Vehicle Classification			
			PC	TT	T	Total			PC	TT	T	Total
Miovision: T2-A	NBL	7:00 am to 7:15 am	8	1	0	9	NBL	11:00 am to 11:15 am	16	0	2	18
	NBT	7:00 am to 7:15 am	32	1	0	33	NBT	11:00 am to 11:15 am	18	4	0	22
	NBR	7:00 am to 7:15 am	16	1	0	17	NBR	11:00 am to 11:15 am	19	1	3	23
	WBL	7:00 am to 7:15 am	22	2	0	24	WBL	11:00 am to 11:15 am	7	3	2	12
	WBT	7:00 am to 7:15 am	30	0	0	30	WBT	11:00 am to 11:15 am	23	0	0	23
	WBR	7:00 am to 7:15 am	5	0	0	5	WBR	11:00 am to 11:15 am	2	0	0	2
	NBL	7:15 am to 7:30 am	26	0	0	26	NBL	11:15 am to 11:30 am	15	0	0	15
	NBT	7:15 am to 7:30 am	38	2	0	40	NBT	11:15 am to 11:30 am	18	7	0	25
	NBR	7:15 am to 7:30 am	23	2	0	25	NBR	11:15 am to 11:30 am	15	4	1	20
	WBL	7:15 am to 7:30 am	29	1	1	31	WBL	11:15 am to 11:30 am	16	5	1	22
	WBT	7:15 am to 7:30 am	63	0	1	64	WBT	11:15 am to 11:30 am	34	1	0	35
	WBR	7:15 am to 7:30 am	6	0	0	6	WBR	11:15 am to 11:30 am	3	0	0	3
	NBL	7:30 am to 7:45 am	48	0	0	48	NBL	11:30 am to 11:45 am	11	0	0	11
	NBT	7:30 am to 7:45 am	52	4	0	56	NBT	11:30 am to 11:45 am	26	4	1	31
	NBR	7:30 am to 7:45 am	14	1	0	15	NBR	11:30 am to 11:45 am	19	3	1	23
	WBL	7:30 am to 7:45 am	51	3	0	54	WBL	11:30 am to 11:45 am	21	3	0	24
	WBT	7:30 am to 7:45 am	99	0	1	100	WBT	11:30 am to 11:45 am	28	0	0	28
	WBR	7:30 am to 7:45 am	9	0	0	9	WBR	11:30 am to 11:45 am	5	0	1	6
	NBL	7:45 am to 8:00 am	29	0	0	29	NBL	11:45 am to 12:00 pm	13	0	0	13
	NBT	7:45 am to 8:00 am	84	3	0	87	NBT	11:45 am to 12:00 pm	36	4	0	40
	NBR	7:45 am to 8:00 am	7	3	1	11	NBR	11:45 am to 12:00 pm	17	5	1	23
	WBL	7:45 am to 8:00 am	51	5	0	56	WBL	11:45 am to 12:00 pm	13	2	0	15
	WBT	7:45 am to 8:00 am	59	0	0	59	WBT	11:45 am to 12:00 pm	27	0	1	28
	WBR	7:45 am to 8:00 am	5	0	0	5	WBR	11:45 am to 12:00 pm	4	0	0	4
	NBL	8:00 am to 8:15 am	17	2	0	19	NBL	12:00 pm to 12:15 pm	19	0	1	20
	NBT	8:00 am to 8:15 am	29	2	0	31	NBT	12:00 pm to 12:15 pm	33	4	0	37
	NBR	8:00 am to 8:15 am	12	4	0	16	NBR	12:00 pm to 12:15 pm	18	8	0	26
	WBL	8:00 am to 8:15 am	23	3	0	26	WBL	12:00 pm to 12:15 pm	12	5	1	18
	WBT	8:00 am to 8:15 am	48	0	0	48	WBT	12:00 pm to 12:15 pm	24	0	0	24
	WBR	8:00 am to 8:15 am	4	0	0	4	WBR	12:00 pm to 12:15 pm	1	0	0	1
	NBL	8:15 am to 8:30 am	14	0	0	14	NBL	12:15 pm to 12:30 pm	13	0	0	13
	NBT	8:15 am to 8:30 am	36	0	0	36	NBT	12:15 pm to 12:30 pm	34	3	0	37
	NBR	8:15 am to 8:30 am	13	6	0	19	NBR	12:15 pm to 12:30 pm	25	3	0	28
	WBL	8:15 am to 8:30 am	17	3	7	27	WBL	12:15 pm to 12:30 pm	18	1	0	19
	WBT	8:15 am to 8:30 am	26	0	1	27	WBT	12:15 pm to 12:30 pm	24	0	0	24
	WBR	8:15 am to 8:30 am	3	0	0	3	WBR	12:15 pm to 12:30 pm	3	0	0	3
	NBL	8:30 am to 8:45 am	13	1	0	14	NBL	12:30 pm to 12:45 pm	11	0	0	11
	NBT	8:30 am to 8:45 am	40	0	0	40	NBT	12:30 pm to 12:45 pm	25	1	1	27
	NBR	8:30 am to 8:45 am	15	2	4	21	NBR	12:30 pm to 12:45 pm	24	8	3	35
	WBL	8:30 am to 8:45 am	20	1	0	21	WBL	12:30 pm to 12:45 pm	20	3	0	23
	WBT	8:30 am to 8:45 am	27	0	1	28	WBT	12:30 pm to 12:45 pm	28	0	0	28
	WBR	8:30 am to 8:45 am	4	0	0	4	WBR	12:30 pm to 12:45 pm	2	0	0	2
	NBL	8:45 am to 9:00 am	16	3	0	19	NBL	12:45 pm to 1:00 pm	17	0	1	18
	NBT	8:45 am to 9:00 am	57	3	0	60	NBT	12:45 pm to 1:00 pm	44	3	2	49
	NBR	8:45 am to 9:00 am	12	3	1	16	NBR	12:45 pm to 1:00 pm	18	2	1	21
	WBL	8:45 am to 9:00 am	15	1	1	17	WBL	12:45 pm to 1:00 pm	12	1	0	13
	WBT	8:45 am to 9:00 am	26	0	0	26	WBT	12:45 pm to 1:00 pm	27	0	1	28
	WBR	8:45 am to 9:00 am	4	0	0	4	WBR	12:45 pm to 1:00 pm	8	0	0	8

Miovision: T2-A	NBL	4:00 pm to 4:15 pm	16	0	0	16
	NBT	4:00 pm to 4:15 pm	24	2	0	26
	NBR	4:00 pm to 4:15 pm	32	4	0	36
	WBL	4:00 pm to 4:15 pm	21	2	1	24
	WBT	4:00 pm to 4:15 pm	42	0	0	42
	WBR	4:00 pm to 4:15 pm	7	0	0	7
	NBL	4:15 pm to 4:30 pm	11	0	0	11
	NBT	4:15 pm to 4:30 pm	22	1	2	25
	NBR	4:15 pm to 4:30 pm	33	2	0	35
	WBL	4:15 pm to 4:30 pm	19	3	1	23
	WBT	4:15 pm to 4:30 pm	36	0	0	36
	WBR	4:15 pm to 4:30 pm	4	0	0	4
	NBL	4:30 pm to 4:45 pm	14	0	0	14
	NBT	4:30 pm to 4:45 pm	33	0	0	33
	NBR	4:30 pm to 4:45 pm	33	1	0	34
	WBL	4:30 pm to 4:45 pm	24	3	0	27
	WBT	4:30 pm to 4:45 pm	32	0	0	32
	WBR	4:30 pm to 4:45 pm	4	0	0	4
	NBL	4:45 pm to 5:00 pm	14	0	1	15
	NBT	4:45 pm to 5:00 pm	39	1	1	41
	NBR	4:45 pm to 5:00 pm	28	1	0	29
	WBL	4:45 pm to 5:00 pm	22	3	2	27
	WBT	4:45 pm to 5:00 pm	34	0	0	34
	WBR	4:45 pm to 5:00 pm	6	0	0	6
	NBL	5:00 pm to 5:15 pm	20	0	0	20
	NBT	5:00 pm to 5:15 pm	43	1	0	44
	NBR	5:00 pm to 5:15 pm	45	3	2	50
	WBL	5:00 pm to 5:15 pm	29	3	0	32
	WBT	5:00 pm to 5:15 pm	33	0	0	33
	WBR	5:00 pm to 5:15 pm	10	0	0	10
	NBL	5:15 pm to 5:30 pm	13	0	0	13
	NBT	5:15 pm to 5:30 pm	44	0	1	45
	NBR	5:15 pm to 5:30 pm	49	2	0	51
	WBL	5:15 pm to 5:30 pm	18	1	0	19
	WBT	5:15 pm to 5:30 pm	38	0	0	38
	WBR	5:15 pm to 5:30 pm	4	0	0	4
	NBL	5:30 pm to 5:45 pm	24	0	1	25
	NBT	5:30 pm to 5:45 pm	28	2	2	32
	NBR	5:30 pm to 5:45 pm	31	0	0	31
	WBL	5:30 pm to 5:45 pm	26	2	3	31
	WBT	5:30 pm to 5:45 pm	28	0	0	28
	WBR	5:30 pm to 5:45 pm	5	0	0	5
	NBL	5:45 pm to 6:00 pm	24	0	0	24
	NBT	5:45 pm to 6:00 pm	33	3	0	36
	NBR	5:45 pm to 6:00 pm	32	1	1	34
	WBL	5:45 pm to 6:00 pm	23	1	3	27
	WBT	5:45 pm to 6:00 pm	30	0	0	30
	WBR	5:45 pm to 6:00 pm	3	0	0	3

Recording Device	Movements	Time	Vehicle Classification				Movements	Time	Vehicle Classification			
			PC	TT	T	Total			PC	TT	T	Total
Miovision: T2-B	EBR	7:00 am to 7:15 am	3	0	0	3	EBR	11:00 am to 11:15 am	15	0	0	15
	EBT	7:00 am to 7:15 am	12	0	0	12	EBT	11:00 am to 11:15 am	32	0	0	32
	EBL	7:00 am to 7:15 am	8	0	0	8	EBL	11:00 am to 11:15 am	11	0	0	11
	SBR	7:00 am to 7:15 am	27	0	1	28	SBR	11:00 am to 11:15 am	14	1	0	15
	SBT	7:00 am to 7:15 am	37	0	0	37	SBT	11:00 am to 11:15 am	26	3	0	29
	SBL	7:00 am to 7:15 am	2	0	0	2	SBL	11:00 am to 11:15 am	2	0	0	2
	EBR	7:15 am to 7:30 am	9	1	0	10	EBR	11:15 am to 11:30 am	23	0	1	24
	EBT	7:15 am to 7:30 am	16	0	1	17	EBT	11:15 am to 11:30 am	36	0	1	37
	EBL	7:15 am to 7:30 am	15	0	0	15	EBL	11:15 am to 11:30 am	18	0	0	18
	SBR	7:15 am to 7:30 am	33	0	0	33	SBR	11:15 am to 11:30 am	29	0	1	30
	SBT	7:15 am to 7:30 am	32	0	2	34	SBT	11:15 am to 11:30 am	26	3	0	29
	SBL	7:15 am to 7:30 am	0	0	0	0	SBL	11:15 am to 11:30 am	4	0	0	4
	EBR	7:30 am to 7:45 am	16	0	0	16	EBR	11:30 am to 11:45 am	15	0	0	15
	EBT	7:30 am to 7:45 am	25	0	1	26	EBT	11:30 am to 11:45 am	31	0	2	33
	EBL	7:30 am to 7:45 am	16	0	0	16	EBL	11:30 am to 11:45 am	16	0	0	16
	SBR	7:30 am to 7:45 am	51	0	0	51	SBR	11:30 am to 11:45 am	18	0	0	18
	SBT	7:30 am to 7:45 am	31	2	5	38	SBT	11:30 am to 11:45 am	34	1	1	36
	SBL	7:30 am to 7:45 am	2	0	0	2	SBL	11:30 am to 11:45 am	2	0	0	2
	EBR	7:45 am to 8:00 am	22	0	1	23	EBR	11:45 am to 12:00 pm	8	0	0	8
	EBT	7:45 am to 8:00 am	31	0	0	31	EBT	11:45 am to 12:00 pm	32	0	0	32
	EBL	7:45 am to 8:00 am	40	0	1	41	EBL	11:45 am to 12:00 pm	28	0	1	29
	SBR	7:45 am to 8:00 am	31	0	0	31	SBR	11:45 am to 12:00 pm	31	0	0	31
	SBT	7:45 am to 8:00 am	23	3	2	28	SBT	11:45 am to 12:00 pm	34	2	1	37
	SBL	7:45 am to 8:00 am	1	0	0	1	SBL	11:45 am to 12:00 pm	2	0	0	2
	EBR	8:00 am to 8:15 am	20	0	0	20	EBR	12:00 pm to 12:15 pm	23	2	0	25
	EBT	8:00 am to 8:15 am	32	0	0	32	EBT	12:00 pm to 12:15 pm	28	0	0	28
	EBL	8:00 am to 8:15 am	32	0	0	32	EBL	12:00 pm to 12:15 pm	24	0	0	24
	SBR	8:00 am to 8:15 am	16	0	0	16	SBR	12:00 pm to 12:15 pm	24	0	0	24
	SBT	8:00 am to 8:15 am	27	1	1	29	SBT	12:00 pm to 12:15 pm	39	1	0	40
	SBL	8:00 am to 8:15 am	3	0	0	3	SBL	12:00 pm to 12:15 pm	7	0	0	7
	EBR	8:15 am to 8:30 am	12	1	0	13	EBR	12:15 pm to 12:30 pm	23	0	0	23
	EBT	8:15 am to 8:30 am	15	0	0	15	EBT	12:15 pm to 12:30 pm	31	0	0	31
	EBL	8:15 am to 8:30 am	16	0	0	16	EBL	12:15 pm to 12:30 pm	22	0	0	22
	SBR	8:15 am to 8:30 am	11	0	1	12	SBR	12:15 pm to 12:30 pm	29	0	0	29
	SBT	8:15 am to 8:30 am	18	6	2	26	SBT	12:15 pm to 12:30 pm	21	5	1	27
	SBL	8:15 am to 8:30 am	0	0	0	0	SBL	12:15 pm to 12:30 pm	1	0	0	1
	EBR	8:30 am to 8:45 am	5	0	3	8	EBR	12:30 pm to 12:45 pm	18	0	1	19
	EBT	8:30 am to 8:45 am	15	0	0	15	EBT	12:30 pm to 12:45 pm	30	0	0	30
	EBL	8:30 am to 8:45 am	10	0	0	10	EBL	12:30 pm to 12:45 pm	25	0	0	25
	SBR	8:30 am to 8:45 am	11	0	0	11	SBR	12:30 pm to 12:45 pm	17	0	0	17
	SBT	8:30 am to 8:45 am	23	1	1	25	SBT	12:30 pm to 12:45 pm	31	5	0	36
	SBL	8:30 am to 8:45 am	0	0	0	0	SBL	12:30 pm to 12:45 pm	3	0	0	3
	EBR	8:45 am to 9:00 am	12	1	1	14	EBR	12:45 pm to 1:00 pm	11	0	1	12
	EBT	8:45 am to 9:00 am	17	0	0	17	EBT	12:45 pm to 1:00 pm	27	1	1	29
	EBL	8:45 am to 9:00 am	11	0	0	11	EBL	12:45 pm to 1:00 pm	23	0	0	23
	SBR	8:45 am to 9:00 am	18	0	0	18	SBR	12:45 pm to 1:00 pm	23	0	0	23
	SBT	8:45 am to 9:00 am	20	4	1	25	SBT	12:45 pm to 1:00 pm	31	6	0	37
	SBL	8:45 am to 9:00 am	0	0	0	0	SBL	12:45 pm to 1:00 pm	3	0	0	3

Miovision: T2-B	EBR	4:00 pm to 4:15 pm	25	1	0	26
	EBT	4:00 pm to 4:15 pm	42	0	0	42
	EBL	4:00 pm to 4:15 pm	26	0	0	26
	SBR	4:00 pm to 4:15 pm	23	0	0	23
	SBT	4:00 pm to 4:15 pm	52	2	0	54
	SBL	4:00 pm to 4:15 pm	8	0	0	8
	EBR	4:15 pm to 4:30 pm	22	0	0	22
	EBT	4:15 pm to 4:30 pm	52	0	1	53
	EBL	4:15 pm to 4:30 pm	29	0	0	29
	SBR	4:15 pm to 4:30 pm	21	0	0	21
	SBT	4:15 pm to 4:30 pm	34	1	1	36
	SBL	4:15 pm to 4:30 pm	2	0	0	2
	EBR	4:30 pm to 4:45 pm	21	0	1	22
	EBT	4:30 pm to 4:45 pm	58	0	0	58
	EBL	4:30 pm to 4:45 pm	24	0	0	24
	SBR	4:30 pm to 4:45 pm	22	0	0	22
	SBT	4:30 pm to 4:45 pm	47	1	1	49
	SBL	4:30 pm to 4:45 pm	4	0	0	4
	EBR	4:45 pm to 5:00 pm	31	0	0	31
	EBT	4:45 pm to 5:00 pm	57	0	0	57
	EBL	4:45 pm to 5:00 pm	29	0	0	29
	SBR	4:45 pm to 5:00 pm	20	0	0	20
	SBT	4:45 pm to 5:00 pm	27	5	0	32
	SBL	4:45 pm to 5:00 pm	5	0	1	6
	EBR	5:00 pm to 5:15 pm	45	0	0	45
	EBT	5:00 pm to 5:15 pm	63	0	1	64
	EBL	5:00 pm to 5:15 pm	50	0	0	50
	SBR	5:00 pm to 5:15 pm	20	0	0	20
	SBT	5:00 pm to 5:15 pm	41	7	0	48
	SBL	5:00 pm to 5:15 pm	10	0	0	10
	EBR	5:15 pm to 5:30 pm	36	0	1	37
	EBT	5:15 pm to 5:30 pm	70	0	0	70
	EBL	5:15 pm to 5:30 pm	37	0	0	37
	SBR	5:15 pm to 5:30 pm	25	0	0	25
	SBT	5:15 pm to 5:30 pm	44	2	1	47
	SBL	5:15 pm to 5:30 pm	2	0	0	2
	EBR	5:30 pm to 5:45 pm	22	0	0	22
	EBT	5:30 pm to 5:45 pm	45	0	0	45
	EBL	5:30 pm to 5:45 pm	34	0	0	34
	SBR	5:30 pm to 5:45 pm	33	0	0	33
	SBT	5:30 pm to 5:45 pm	40	1	0	41
	SBL	5:30 pm to 5:45 pm	4	0	0	4
	EBR	5:45 pm to 6:00 pm	26	1	0	27
	EBT	5:45 pm to 6:00 pm	52	0	0	52
	EBL	5:45 pm to 6:00 pm	28	0	0	28
	SBR	5:45 pm to 6:00 pm	24	0	0	24
	SBT	5:45 pm to 6:00 pm	34	1	0	35
	SBL	5:45 pm to 6:00 pm	4	0	0	4

A.1.9 Roundabout at Kearney

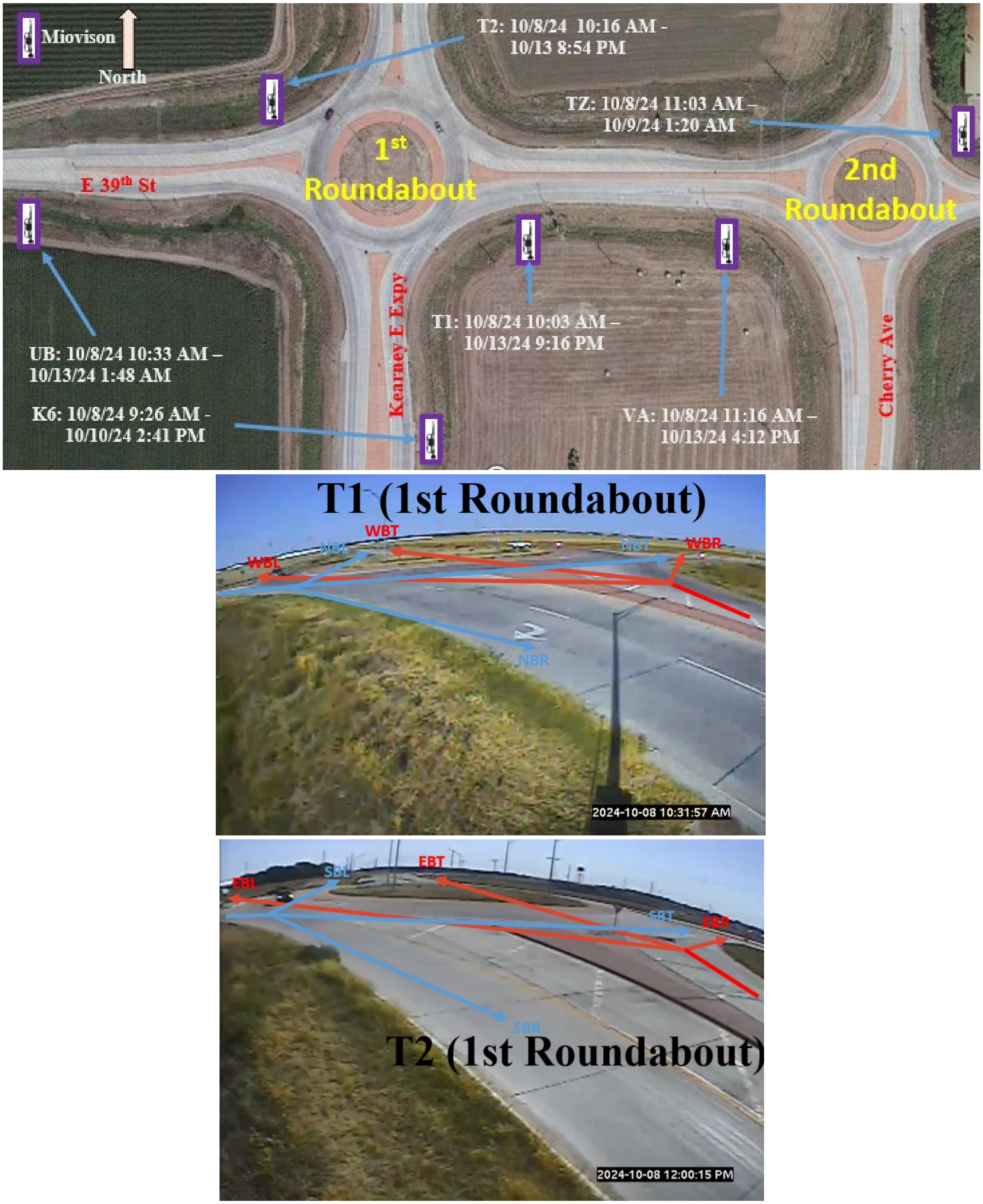


Figure A.9 Data collection device locations and corresponding traffic movements at the Kearney site

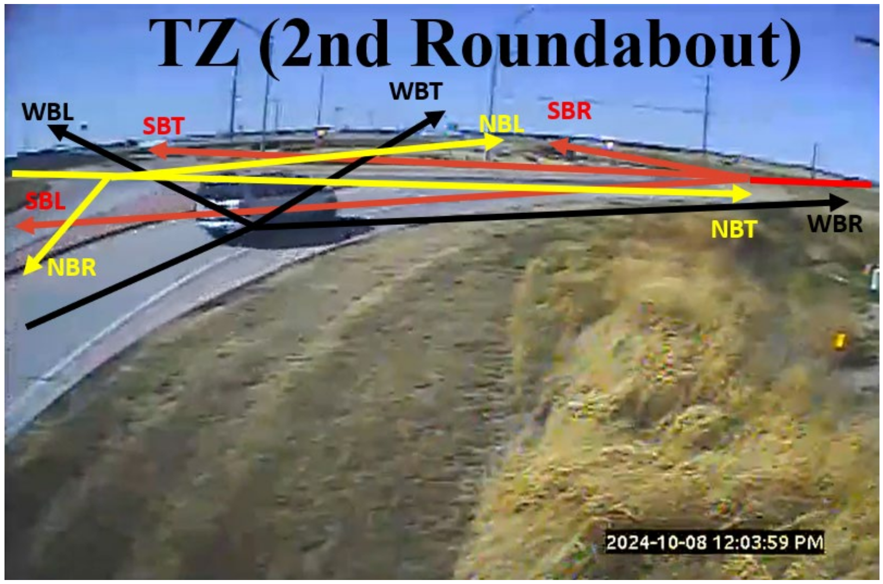


Figure A.9 cont. Data collection device locations and corresponding traffic movements at the Kearney site

Table A.9 Traffic volume counts at the Kearney site

Date: 10/8/2024 (4pm to 6pm), 10/9/2024 (7am to 9am), 10/9/2024 (11am to 1pm)

Recording Device	Movements	Time	Vehicle Classification				Movements	Time	Vehicle Classification			
			PC	TT	T	Total			PC	TT	T	Total
Miovision: T1	NBL	4:00 pm to 4:15 pm	22	0	0	22	NBL	7:00 am to 7:15 am	10	0	0	10
	NBT	4:00 pm to 4:15 pm	14	2	0	16	NBT	7:00 am to 7:15 am	11	1	0	12
	NBR	4:00 pm to 4:15 pm	6	3	0	9	NBR	7:00 am to 7:15 am	14	2	0	16
	WBL	4:00 pm to 4:15 pm	10	0	0	10	WBL	7:00 am to 7:15 am	14	2	0	16
	WBT	4:00 pm to 4:15 pm	32	0	1	33	WBT	7:00 am to 7:15 am	28	0	0	28
	WBR	4:00 pm to 4:15 pm	13	0	0	13	WBR	7:00 am to 7:15 am	8	1	0	9
	NBL	4:15 pm to 4:30 pm	14	1	0	15	NBL	7:15 am to 7:30 am	22	1	0	23
	NBT	4:15 pm to 4:30 pm	17	1	0	18	NBT	7:15 am to 7:30 am	9	0	0	9
	NBR	4:15 pm to 4:30 pm	11	2	0	13	NBR	7:15 am to 7:30 am	12	1	1	14
	WBL	4:15 pm to 4:30 pm	12	2	2	16	WBL	7:15 am to 7:30 am	4	0	0	4
	WBT	4:15 pm to 4:30 pm	25	0	0	25	WBT	7:15 am to 7:30 am	10	0	0	10
	WBR	4:15 pm to 4:30 pm	6	0	0	6	WBR	7:15 am to 7:30 am	7	1	0	8
	NBL	4:30 pm to 4:45 pm	36	0	1	37	NBL	7:30 am to 7:45 am	35	0	0	35
	NBT	4:30 pm to 4:45 pm	21	1	0	22	NBT	7:30 am to 7:45 am	25	4	0	29
	NBR	4:30 pm to 4:45 pm	17	3	0	20	NBR	7:30 am to 7:45 am	23	0	1	24
	WBL	4:30 pm to 4:45 pm	24	5	0	29	WBL	7:30 am to 7:45 am	11	3	0	14
	WBT	4:30 pm to 4:45 pm	58	2	1	61	WBT	7:30 am to 7:45 am	18	0	0	18
	WBR	4:30 pm to 4:45 pm	14	0	0	14	WBR	7:30 am to 7:45 am	6	0	0	6
	NBL	4:45 pm to 5:00 pm	23	2	0	25	NBL	7:45 am to 8:00 am	24	3	0	27
	NBT	4:45 pm to 5:00 pm	32	0	1	33	NBT	7:45 am to 8:00 am	43	1	2	46
	NBR	4:45 pm to 5:00 pm	15	3	2	20	NBR	7:45 am to 8:00 am	26	6	0	32
	WBL	4:45 pm to 5:00 pm	8	1	0	9	WBL	7:45 am to 8:00 am	8	1	0	9
	WBT	4:45 pm to 5:00 pm	24	2	0	26	WBT	7:45 am to 8:00 am	23	0	2	25
	WBR	4:45 pm to 5:00 pm	9	1	0	10	WBR	7:45 am to 8:00 am	8	2	0	10
	NBL	5:00 pm to 5:15 pm	18	0	0	18	NBL	8:00 am to 8:15 am	17	1	0	18
	NBT	5:00 pm to 5:15 pm	25	1	0	26	NBT	8:00 am to 8:15 am	13	1	0	14
	NBR	5:00 pm to 5:15 pm	7	1	0	8	NBR	8:00 am to 8:15 am	20	3	0	23
	WBL	5:00 pm to 5:15 pm	19	2	0	21	WBL	8:00 am to 8:15 am	5	1	1	7
	WBT	5:00 pm to 5:15 pm	48	0	0	48	WBT	8:00 am to 8:15 am	16	1	0	17
	WBR	5:00 pm to 5:15 pm	15	2	0	17	WBR	8:00 am to 8:15 am	8	0	0	8
	NBL	5:15 pm to 5:30 pm	23	0	0	23	NBL	8:15 am to 8:30 am	13	1	1	15
	NBT	5:15 pm to 5:30 pm	18	0	0	18	NBT	8:15 am to 8:30 am	12	1	1	14
	NBR	5:15 pm to 5:30 pm	12	2	0	14	NBR	8:15 am to 8:30 am	5	3	0	8
	WBL	5:15 pm to 5:30 pm	27	1	0	28	WBL	8:15 am to 8:30 am	11	5	0	16
	WBT	5:15 pm to 5:30 pm	37	0	0	37	WBT	8:15 am to 8:30 am	16	1	1	18
	WBR	5:15 pm to 5:30 pm	13	0	0	13	WBR	8:15 am to 8:30 am	1	0	0	1
	NBL	5:30 pm to 5:45 pm	14	1	0	15	NBL	8:30 am to 8:45 am	17	1	0	18
	NBT	5:30 pm to 5:45 pm	14	1	1	16	NBT	8:30 am to 8:45 am	15	1	0	16
	NBR	5:30 pm to 5:45 pm	10	1	0	11	NBR	8:30 am to 8:45 am	8	4	2	14
	WBL	5:30 pm to 5:45 pm	12	1	0	13	WBL	8:30 am to 8:45 am	9	3	1	13
	WBT	5:30 pm to 5:45 pm	34	0	0	34	WBT	8:30 am to 8:45 am	23	1	0	24
	WBR	5:30 pm to 5:45 pm	8	0	0	8	WBR	8:30 am to 8:45 am	3	0	0	3
	NBL	5:45 pm to 6:00 pm	15	0	0	15	NBL	8:45 am to 9:00 am	19	2	0	21
	NBT	5:45 pm to 6:00 pm	22	1	0	23	NBT	8:45 am to 9:00 am	10	1	1	12
	NBR	5:45 pm to 6:00 pm	7	3	1	11	NBR	8:45 am to 9:00 am	10	5	0	15
	WBL	5:45 pm to 6:00 pm	11	2	0	13	WBL	8:45 am to 9:00 am	9	2	1	12
	WBT	5:45 pm to 6:00 pm	13	0	0	13	WBT	8:45 am to 9:00 am	26	2	2	30
	WBR	5:45 pm to 6:00 pm	3	1	0	4	WBR	8:45 am to 9:00 am	2	0	1	3

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T1	NBL	11:00 am to 11:15 am	12	2	0	14
	NBT	11:00 am to 11:15 am	13	1	0	14
	NBR	11:00 am to 11:15 am	5	2	0	7
	WBL	11:00 am to 11:15 am	8	4	0	12
	WBT	11:00 am to 11:15 am	17	3	0	20
	WBR	11:00 am to 11:15 am	4	0	1	5
	NBL	11:15 am to 11:30 am	9	2	0	11
	NBT	11:15 am to 11:30 am	8	1	1	10
	NBR	11:15 am to 11:30 am	2	1	0	3
	WBL	11:15 am to 11:30 am	12	2	0	14
	WBT	11:15 am to 11:30 am	19	0	0	19
	WBR	11:15 am to 11:30 am	5	2	0	7
	NBL	11:30 am to 11:45 am	23	0	0	23
	NBT	11:30 am to 11:45 am	18	2	1	21
	NBR	11:30 am to 11:45 am	20	4	0	24
	WBL	11:30 am to 11:45 am	9	3	0	12
	WBT	11:30 am to 11:45 am	20	1	0	21
	WBR	11:30 am to 11:45 am	5	0	3	8
	NBL	11:45 am to 12:00 pm	14	1	1	16
	NBT	11:45 am to 12:00 pm	10	3	1	14
	NBR	11:45 am to 12:00 pm	12	2	1	15
	WBL	11:45 am to 12:00 pm	24	1	0	25
	WBT	11:45 am to 12:00 pm	26	0	1	27
	WBR	11:45 am to 12:00 pm	4	0	0	4
	NBL	12:00 pm to 12:15 pm	8	1	1	10
	NBT	12:00 pm to 12:15 pm	7	1	0	8
	NBR	12:00 pm to 12:15 pm	11	1	0	12
	WBL	12:00 pm to 12:15 pm	21	2	2	25
	WBT	12:00 pm to 12:15 pm	42	1	0	43
	WBR	12:00 pm to 12:15 pm	13	0	0	13
	NBL	12:15 pm to 12:30 pm	9	1	0	10
	NBT	12:15 pm to 12:30 pm	10	2	1	13
	NBR	12:15 pm to 12:30 pm	4	3	0	7
	WBL	12:15 pm to 12:30 pm	8	1	0	9
	WBT	12:15 pm to 12:30 pm	25	0	0	25
	WBR	12:15 pm to 12:30 pm	4	0	0	4
	NBL	12:30 pm to 12:45 pm	10	1	0	11
	NBT	12:30 pm to 12:45 pm	15	0	2	17
	NBR	12:30 pm to 12:45 pm	12	4	0	16
	WBL	12:30 pm to 12:45 pm	7	4	1	12
	WBT	12:30 pm to 12:45 pm	20	0	2	22
	WBR	12:30 pm to 12:45 pm	7	1	0	8
	NBL	12:45 pm to 1:00 pm	11	0	1	12
	NBT	12:45 pm to 1:00 pm	15	2	1	18
	NBR	12:45 pm to 1:00 pm	12	3	0	15
	WBL	12:45 pm to 1:00 pm	7	4	0	11
	WBT	12:45 pm to 1:00 pm	14	1	0	15
	WBR	12:45 pm to 1:00 pm	2	2	0	4

Recording Device	Movements	Time	Vehicle Classification				Movements	Time	Vehicle Classification			
			PC	TT	T	Total			PC	TT	T	Total
Miovision: T2	SBL	4:00 pm to 4:15 pm	5	1	0	6	SBL	7:00 am to 7:15 am	15	0	0	15
	SBT	4:00 pm to 4:15 pm	16	2	0	18	SBT	7:00 am to 7:15 am	21	1	1	23
	SBR	4:00 pm to 4:15 pm	1	0	0	1	SBR	7:00 am to 7:15 am	2	0	0	2
	EBL	4:00 pm to 4:15 pm	5	0	0	5	EBL	7:00 am to 7:15 am	2	0	0	2
	EBT	4:00 pm to 4:15 pm	30	0	3	33	EBT	7:00 am to 7:15 am	24	0	1	25
	EBR	4:00 pm to 4:15 pm	25	1	0	26	EBR	7:00 am to 7:15 am	24	0	0	24
	SBL	4:15 pm to 4:30 pm	6	1	0	7	SBL	7:15 am to 7:30 am	12	1	0	13
	SBT	4:15 pm to 4:30 pm	15	2	1	18	SBT	7:15 am to 7:30 am	11	0	0	11
	SBR	4:15 pm to 4:30 pm	1	0	0	1	SBR	7:15 am to 7:30 am	1	0	0	1
	EBL	4:15 pm to 4:30 pm	1	0	0	1	EBL	7:15 am to 7:30 am	4	0	0	4
	EBT	4:15 pm to 4:30 pm	17	1	0	18	EBT	7:15 am to 7:30 am	24	0	0	24
	EBR	4:15 pm to 4:30 pm	15	1	2	18	EBR	7:15 am to 7:30 am	22	1	1	24
	SBL	4:30 pm to 4:45 pm	4	1	0	5	SBL	7:30 am to 7:45 am	12	0	0	12
	SBT	4:30 pm to 4:45 pm	23	0	1	24	SBT	7:30 am to 7:45 am	20	1	0	21
	SBR	4:30 pm to 4:45 pm	9	0	0	9	SBR	7:30 am to 7:45 am	1	1	0	2
	EBL	4:30 pm to 4:45 pm	2	0	0	2	EBL	7:30 am to 7:45 am	7	0	0	7
	EBT	4:30 pm to 4:45 pm	20	1	1	22	EBT	7:30 am to 7:45 am	37	1	0	38
	EBR	4:30 pm to 4:45 pm	15	0	0	15	EBR	7:30 am to 7:45 am	13	0	0	13
	SBL	4:45 pm to 5:00 pm	6	2	0	8	SBL	7:45 am to 8:00 am	26	1	0	27
	SBT	4:45 pm to 5:00 pm	23	0	1	24	SBT	7:45 am to 8:00 am	16	0	0	16
	SBR	4:45 pm to 5:00 pm	6	0	0	6	SBR	7:45 am to 8:00 am	1	0	0	1
	EBL	4:45 pm to 5:00 pm	2	0	0	2	EBL	7:45 am to 8:00 am	10	0	0	10
	EBT	4:45 pm to 5:00 pm	21	2	0	23	EBT	7:45 am to 8:00 am	40	0	0	40
	EBR	4:45 pm to 5:00 pm	20	0	1	21	EBR	7:45 am to 8:00 am	14	1	1	16
	SBL	5:00 pm to 5:15 pm	10	1	0	11	SBL	8:00 am to 8:15 am	3	0	0	3
	SBT	5:00 pm to 5:15 pm	37	1	0	38	SBT	8:00 am to 8:15 am	20	1	0	21
	SBR	5:00 pm to 5:15 pm	7	0	0	7	SBR	8:00 am to 8:15 am	2	0	0	2
	EBL	5:00 pm to 5:15 pm	2	0	0	2	EBL	8:00 am to 8:15 am	2	0	0	2
	EBT	5:00 pm to 5:15 pm	29	0	0	29	EBT	8:00 am to 8:15 am	19	0	0	19
	EBR	5:00 pm to 5:15 pm	21	1	0	22	EBR	8:00 am to 8:15 am	19	0	0	19
	SBL	5:15 pm to 5:30 pm	5	1	0	6	SBL	8:15 am to 8:30 am	5	0	0	5
	SBT	5:15 pm to 5:30 pm	22	0	0	22	SBT	8:15 am to 8:30 am	15	0	2	17
	SBR	5:15 pm to 5:30 pm	1	0	0	1	SBR	8:15 am to 8:30 am	0	0	0	0
	EBL	5:15 pm to 5:30 pm	1	0	0	1	EBL	8:15 am to 8:30 am	3	0	0	3
	EBT	5:15 pm to 5:30 pm	18	0	1	19	EBT	8:15 am to 8:30 am	17	1	0	18
	EBR	5:15 pm to 5:30 pm	17	2	0	19	EBR	8:15 am to 8:30 am	10	1	0	11
	SBL	5:30 pm to 5:45 pm	8	0	0	8	SBL	8:30 am to 8:45 am	6	0	0	6
	SBT	5:30 pm to 5:45 pm	17	1	0	18	SBT	8:30 am to 8:45 am	14	0	1	15
	SBR	5:30 pm to 5:45 pm	2	0	0	2	SBR	8:30 am to 8:45 am	3	1	0	4
	EBL	5:30 pm to 5:45 pm	1	0	0	1	EBL	8:30 am to 8:45 am	4	0	0	4
	EBT	5:30 pm to 5:45 pm	17	0	0	17	EBT	8:30 am to 8:45 am	12	0	1	13
	EBR	5:30 pm to 5:45 pm	13	0	0	13	EBR	8:30 am to 8:45 am	12	0	0	12
	SBL	5:45 pm to 6:00 pm	3	0	0	3	SBL	8:45 am to 9:00 am	2	0	1	3
	SBT	5:45 pm to 6:00 pm	16	0	0	16	SBT	8:45 am to 9:00 am	11	2	0	13
	SBR	5:45 pm to 6:00 pm	3	0	0	3	SBR	8:45 am to 9:00 am	2	0	1	3
	EBL	5:45 pm to 6:00 pm	1	0	0	1	EBL	8:45 am to 9:00 am	1	0	0	1
	EBT	5:45 pm to 6:00 pm	9	0	0	9	EBT	8:45 am to 9:00 am	10	3	0	13
	EBR	5:45 pm to 6:00 pm	17	0	0	17	EBR	8:45 am to 9:00 am	16	0	0	16

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	SBL	11:00 am to 11:15 am	3	0	0	3
	SBT	11:00 am to 11:15 am	13	0	0	13
	SBR	11:00 am to 11:15 am	0	0	0	0
	EBL	11:00 am to 11:15 am	1	0	0	1
	EBT	11:00 am to 11:15 am	14	1	2	17
	EBR	11:00 am to 11:15 am	11	0	0	11
	SBL	11:15 am to 11:30 am	3	0	2	5
	SBT	11:15 am to 11:30 am	11	0	1	12
	SBR	11:15 am to 11:30 am	2	0	0	2
	EBL	11:15 am to 11:30 am	0	0	0	0
	EBT	11:15 am to 11:30 am	12	2	2	16
	EBR	11:15 am to 11:30 am	9	0	1	10
	SBL	11:30 am to 11:45 am	7	0	0	7
	SBT	11:30 am to 11:45 am	14	0	0	14
	SBR	11:30 am to 11:45 am	2	0	1	3
	EBL	11:30 am to 11:45 am	2	0	0	2
	EBT	11:30 am to 11:45 am	14	0	0	14
	EBR	11:30 am to 11:45 am	11	2	0	13
	SBL	11:45 am to 12:00 pm	5	0	0	5
	SBT	11:45 am to 12:00 pm	17	3	0	20
	SBR	11:45 am to 12:00 pm	3	0	0	3
	EBL	11:45 am to 12:00 pm	1	0	0	1
	EBT	11:45 am to 12:00 pm	14	1	1	16
	EBR	11:45 am to 12:00 pm	15	1	0	16
	SBL	12:00 pm to 12:15 pm	8	0	3	11
	SBT	12:00 pm to 12:15 pm	9	2	2	13
	SBR	12:00 pm to 12:15 pm	3	0	0	3
	EBL	12:00 pm to 12:15 pm	2	0	0	2
	EBT	12:00 pm to 12:15 pm	24	0	1	25
	EBR	12:00 pm to 12:15 pm	14	1	1	16
	SBL	12:15 pm to 12:30 pm	10	0	0	10
	SBT	12:15 pm to 12:30 pm	14	1	0	15
	SBR	12:15 pm to 12:30 pm	0	0	0	0
	EBL	12:15 pm to 12:30 pm	0	0	0	0
	EBT	12:15 pm to 12:30 pm	25	1	0	26
	EBR	12:15 pm to 12:30 pm	9	0	1	10
	SBL	12:30 pm to 12:45 pm	5	0	0	5
	SBT	12:30 pm to 12:45 pm	8	2	0	10
	SBR	12:30 pm to 12:45 pm	4	0	0	4
	EBL	12:30 pm to 12:45 pm	2	0	1	3
	EBT	12:30 pm to 12:45 pm	30	1	0	31
	EBR	12:30 pm to 12:45 pm	10	3	2	15
	SBL	12:45 pm to 1:00 pm	5	0	0	5
	SBT	12:45 pm to 1:00 pm	11	1	1	13
	SBR	12:45 pm to 1:00 pm	4	0	0	4
	EBL	12:45 pm to 1:00 pm	3	0	0	3
	EBT	12:45 pm to 1:00 pm	32	0	0	32
	EBR	12:45 pm to 1:00 pm	13	0	0	13

Recording Device	Movements	Time	Vehicle Classification				Movements	Time	Vehicle Classification			
			PC	TT	T	Total			PC	TT	T	Total
Miovision: VA	EBL	4:00 pm to 4:15 pm	3	0	1	4	EBL	7:00 am to 7:15 am	5	0	0	5
	EBT	4:00 pm to 4:15 pm	13	1	1	15	EBT	7:00 am to 7:15 am	21	0	2	23
	EBR	4:00 pm to 4:15 pm	26	3	1	30	EBR	7:00 am to 7:15 am	25	1	0	26
	EBL	4:15 pm to 4:30 pm	0	0	0	0	EBL	7:15 am to 7:30 am	6	0	0	6
	EBT	4:15 pm to 4:30 pm	7	2	0	9	EBT	7:15 am to 7:30 am	16	0	1	17
	EBR	4:15 pm to 4:30 pm	26	2	0	28	EBR	7:15 am to 7:30 am	27	2	1	30
	EBL	4:30 pm to 4:45 pm	1	0	0	1	EBL	7:30 am to 7:45 am	11	0	0	11
	EBT	4:30 pm to 4:45 pm	4	2	0	6	EBT	7:30 am to 7:45 am	21	0	0	21
	EBR	4:30 pm to 4:45 pm	42	3	1	46	EBR	7:30 am to 7:45 am	39	1	1	41
	EBL	4:45 pm to 5:00 pm	0	0	0	0	EBL	7:45 am to 8:00 am	14	1	0	15
	EBT	4:45 pm to 5:00 pm	4	2	1	7	EBT	7:45 am to 8:00 am	46	1	0	47
	EBR	4:45 pm to 5:00 pm	35	5	2	42	EBR	7:45 am to 8:00 am	32	5	1	38
	EBL	5:00 pm to 5:15 pm	2	0	0	2	EBL	8:00 am to 8:15 am	10	1	0	11
	EBT	5:00 pm to 5:15 pm	15	0	0	15	EBT	8:00 am to 8:15 am	14	1	0	15
	EBR	5:00 pm to 5:15 pm	31	2	0	33	EBR	8:00 am to 8:15 am	20	2	1	23
	EBL	5:15 pm to 5:30 pm	3	0	1	4	EBL	8:15 am to 8:30 am	4	1	0	5
	EBT	5:15 pm to 5:30 pm	6	0	1	7	EBT	8:15 am to 8:30 am	12	2	0	14
	EBR	5:15 pm to 5:30 pm	23	4	0	27	EBR	8:15 am to 8:30 am	11	1	0	12
	EBL	5:30 pm to 5:45 pm	2	0	0	2	EBL	8:30 am to 8:45 am	2	0	0	2
	EBT	5:30 pm to 5:45 pm	6	0	0	6	EBT	8:30 am to 8:45 am	6	1	2	9
	EBR	5:30 pm to 5:45 pm	26	0	0	26	EBR	8:30 am to 8:45 am	17	4	1	22
	EBL	5:45 pm to 6:00 pm	0	0	0	0	EBL	8:45 am to 9:00 am	7	2	1	10
	EBT	5:45 pm to 6:00 pm	2	0	0	2	EBT	8:45 am to 9:00 am	3	1	2	6
	EBR	5:45 pm to 6:00 pm	17	3	1	21	EBR	8:45 am to 9:00 am	12	1	0	13

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: VA	EBR	11:00 am to 11:15 am	3	0	0	3
	EBT	11:00 am to 11:15 am	5	1	1	7
	EBL	11:00 am to 11:15 am	15	2	0	17
	EBR	11:15 am to 11:30 am	2	1	2	5
	EBT	11:15 am to 11:30 am	1	0	1	2
	EBL	11:15 am to 11:30 am	14	2	1	17
	EBR	11:30 am to 11:45 am	4	0	0	4
	EBT	11:30 am to 11:45 am	5	0	0	5
	EBL	11:30 am to 11:45 am	30	5	0	35
	EBR	11:45 am to 12:00 pm	2	0	0	2
	EBT	11:45 am to 12:00 pm	12	0	0	12
	EBL	11:45 am to 12:00 pm	19	3	1	23
	EBR	12:00 pm to 12:15 pm	2	1	0	3
	EBT	12:00 pm to 12:15 pm	18	0	0	18
	EBL	12:00 pm to 12:15 pm	24	0	3	27
	EBR	12:15 pm to 12:30 pm	2	1	0	3
	EBT	12:15 pm to 12:30 pm	13	1	0	14
	EBL	12:15 pm to 12:30 pm	23	2	0	25
	EBR	12:30 pm to 12:45 pm	9	1	0	10
	EBT	12:30 pm to 12:45 pm	17	1	0	18
	EBL	12:30 pm to 12:45 pm	21	3	0	24
	EBR	12:45 pm to 1:00 pm	7	0	0	7
	EBT	12:45 pm to 1:00 pm	32	1	0	33
	EBL	12:45 pm to 1:00 pm	13	2	0	15

Recording Device	Movements	Time	Vehicle Classification				Movements	Time	Vehicle Classification			
			PC	TT	T	Total			PC	TT	T	Total
Miovision: TZ	NBL	4:00 pm to 4:15 pm	17	1	0	18	NBL	5:00 pm to 5:15 pm	18	2	0	20
	NBT	4:00 pm to 4:15 pm	2	0	0	2	NBT	5:00 pm to 5:15 pm	0	0	0	0
	NBR	4:00 pm to 4:15 pm	4	0	0	4	NBR	5:00 pm to 5:15 pm	0	0	0	0
	WBL	4:00 pm to 4:15 pm	3	0	0	3	WBL	5:00 pm to 5:15 pm	22	0	0	22
	WBT	4:00 pm to 4:15 pm	32	1	0	33	WBT	5:00 pm to 5:15 pm	60	2	0	62
	WBR	4:00 pm to 4:15 pm	1	0	0	1	WBR	5:00 pm to 5:15 pm	0	0	0	0
	SBL	4:00 pm to 4:15 pm	1	0	0	1	SBL	5:00 pm to 5:15 pm	1	0	0	1
	SBT	4:00 pm to 4:15 pm	3	0	0	3	SBT	5:00 pm to 5:15 pm	0	0	0	0
	SBR	4:00 pm to 4:15 pm	6	0	0	6	SBR	5:00 pm to 5:15 pm	7	0	0	7
	NBL	4:15 pm to 4:30 pm	18	2	1	21	NBL	5:15 pm to 5:30 pm	33	1	0	34
	NBT	4:15 pm to 4:30 pm	0	0	0	0	NBT	5:15 pm to 5:30 pm	1	0	0	1
	NBR	4:15 pm to 4:30 pm	1	0	1	2	NBR	5:15 pm to 5:30 pm	0	0	0	0
	WBL	4:15 pm to 4:30 pm	1	0	0	1	WBL	5:15 pm to 5:30 pm	1	0	0	1
	WBT	4:15 pm to 4:30 pm	21	0	0	21	WBT	5:15 pm to 5:30 pm	37	0	0	37
	WBR	4:15 pm to 4:30 pm	1	0	0	1	WBR	5:15 pm to 5:30 pm	0	0	0	0
	SBL	4:15 pm to 4:30 pm	0	0	0	0	SBL	5:15 pm to 5:30 pm	0	0	0	0
	SBT	4:15 pm to 4:30 pm	1	1	0	2	SBT	5:15 pm to 5:30 pm	3	0	0	3
	SBR	4:15 pm to 4:30 pm	4	1	1	6	SBR	5:15 pm to 5:30 pm	6	0	0	6
	NBL	4:30 pm to 4:45 pm	12	3	0	15	NBL	5:30 pm to 5:45 pm	25	1	1	27
	NBT	4:30 pm to 4:45 pm	0	0	0	0	NBT	5:30 pm to 5:45 pm	0	0	0	0
	NBR	4:30 pm to 4:45 pm	1	0	0	1	NBR	5:30 pm to 5:45 pm	0	0	0	0
	WBL	4:30 pm to 4:45 pm	12	0	0	12	WBL	5:30 pm to 5:45 pm	0	0	0	0
	WBT	4:30 pm to 4:45 pm	76	2	0	78	WBT	5:30 pm to 5:45 pm	23	0	0	23
	WBR	4:30 pm to 4:45 pm	0	0	0	0	WBR	5:30 pm to 5:45 pm	0	0	0	0
	SBL	4:30 pm to 4:45 pm	1	0	0	1	SBL	5:30 pm to 5:45 pm	0	0	0	0
	SBT	4:30 pm to 4:45 pm	2	0	0	2	SBT	5:30 pm to 5:45 pm	2	0	0	2
	SBR	4:30 pm to 4:45 pm	11	2	0	13	SBR	5:30 pm to 5:45 pm	2	0	0	2
	NBL	4:45 pm to 5:00 pm	15	1	0	16	NBL	5:45 pm to 6:00 pm	11	3	0	14
	NBT	4:45 pm to 5:00 pm	1	0	0	1	NBT	5:45 pm to 6:00 pm	1	0	0	1
	NBR	4:45 pm to 5:00 pm	2	0	0	2	NBR	5:45 pm to 6:00 pm	2	0	0	2
	WBL	4:45 pm to 5:00 pm	0	1	0	1	WBL	5:45 pm to 6:00 pm	0	0	0	0
	WBT	4:45 pm to 5:00 pm	21	2	0	23	WBT	5:45 pm to 6:00 pm	7	0	0	7
	WBR	4:45 pm to 5:00 pm	0	0	0	0	WBR	5:45 pm to 6:00 pm	0	0	0	0
	SBL	4:45 pm to 5:00 pm	0	0	0	0	SBL	5:45 pm to 6:00 pm	0	0	0	0
	SBT	4:45 pm to 5:00 pm	8	0	0	8	SBT	5:45 pm to 6:00 pm	3	0	0	3
	SBR	4:45 pm to 5:00 pm	4	1	0	5	SBR	5:45 pm to 6:00 pm	4	0	0	4

A.2 Interchange Volumes

A.2.1 DIstop at Fremont

Figure A.10 Data collection device locations and corresponding traffic movements at the Fremont site

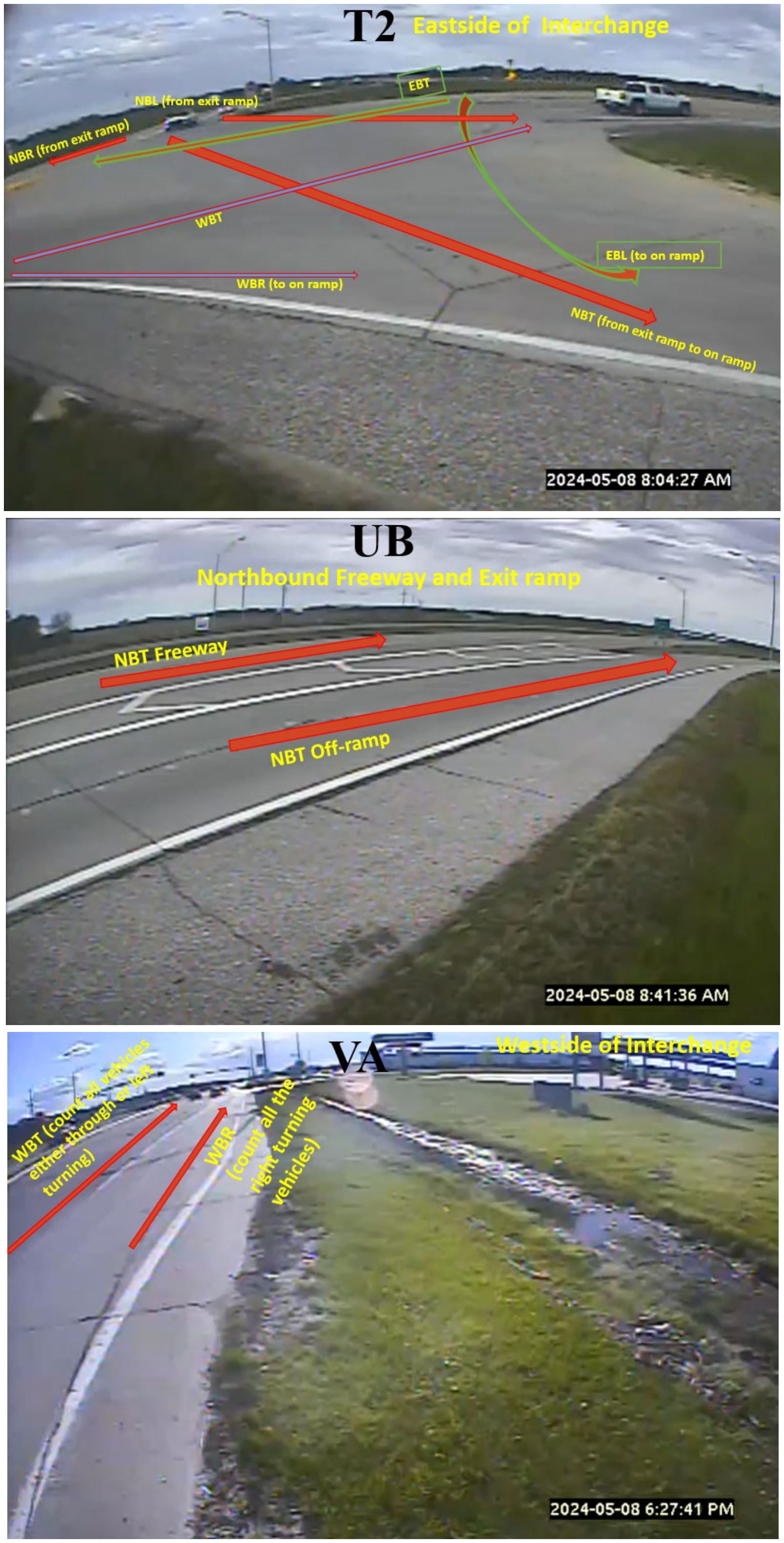


Figure A.10 cont. Data collection device locations and corresponding traffic movements at the Fremont site

Table A.10 Traffic volume counts at the Fremont site

Date: 5/8/2024

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T1	EBT	8:00 am to 8:15 am	67	5	0	72
	WBT	8:00 am to 8:15 am	67	9	1	77
	EBT	8:15 am to 8:30 am	35	5	0	40
	WBT	8:15 am to 8:30 am	73	4	1	78
	EBT	8:30 am to 8:45 am	41	13	0	54
	WBT	8:30 am to 8:45 am	59	5	1	65
	EBT	8:45 am to 9:00 am	43	6	1	50
	WBT	8:45 am to 9:00 am	70	8	1	79
	EBT	9:00 am to 9:15 am	36	7	1	44
	WBT	9:00 am to 9:15 am	58	6	4	68
	EBT	9:15 am to 9:30 am	35	7	1	43
	WBT	9:15 am to 9:30 am	43	9	1	53
	EBT	9:30 am to 9:45 am	41	5	1	47
	WBT	9:30 am to 9:45 am	46	6	2	54
	EBT	9:45 am to 10:00 am	53	7	1	61
	WBT	9:45 am to 10:00 am	54	6	0	60
	EBT	4:00 pm to 4:15 pm	89	12	0	101
	WBT	4:00 pm to 4:15 pm	96	3	0	99
	EBT	4:15 pm to 4:30 pm	94	4	0	98
	WBT	4:15 pm to 4:30 pm	66	6	0	72
	EBT	4:30 pm to 4:45 pm	81	1	0	82
	WBT	4:30 pm to 4:45 pm	67	5	1	73
	EBT	4:45 pm to 5:00 pm	101	1	0	102
	WBT	4:45 pm to 5:00 pm	103	6	0	109
	EBT	5:00 pm to 5:15 pm	105	2	0	107
	WBT	5:00 pm to 5:15 pm	136	5	0	141
	EBT	5:15 pm to 5:30 pm	99	3	1	103
	WBT	5:15 pm to 5:30 pm	84	3	0	87
	EBT	5:30 pm to 5:45 pm	88	3	0	91
	WBT	5:30 pm to 5:45 pm	76	2	0	78
	EBT	5:45 pm to 6:00 pm	72	1	0	73
	WBT	5:45 pm to 6:00 pm	57	4	0	61

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	WBT	8:00 am to 8:15 am	52	5	1	58
	WBR	8:00 am to 8:15 am	5	3	0	8
	EBT	8:00 am to 8:15 am	34	0	0	34
	EBL	8:00 am to 8:15 am	7	0	0	7
	NBT	8:00 am to 8:15 am	0	0	0	0
	NBR	8:00 am to 8:15 am	15	3	0	18
	NBL	8:00 am to 8:15 am	49	2	2	53
	WBT	8:15 am to 8:30 am	62	2	1	65
	WBR	8:15 am to 8:30 am	8	0	0	8
	EBT	8:15 am to 8:30 am	21	3	0	24
	EBL	8:15 am to 8:30 am	4	2	0	6
	NBT	8:15 am to 8:30 am	0	0	0	0
	NBR	8:15 am to 8:30 am	15	3	0	18
	NBL	8:15 am to 8:30 am	68	1	0	69
	WBT	8:30 am to 8:45 am	56	4	4	64
	WBR	8:30 am to 8:45 am	6	2	2	10
	EBT	8:30 am to 8:45 am	29	6	6	41
	EBL	8:30 am to 8:45 am	4	0	0	4
	NBT	8:30 am to 8:45 am	0	0	0	0
	NBR	8:30 am to 8:45 am	10	5	5	20
	NBL	8:30 am to 8:45 am	67	0	0	67
	WBT	8:45 am to 9:00 am	59	4	1	64
	WBR	8:45 am to 9:00 am	7	3	0	10
	EBT	8:45 am to 9:00 am	35	3	0	38
	EBL	8:45 am to 9:00 am	6	0	0	6
	NBT	8:45 am to 9:00 am	0	0	0	0
	NBR	8:45 am to 9:00 am	90	3	1	94
	NBL	8:45 am to 9:00 am	60	0	2	62
	WBT	9:00 am to 9:15 am	59	3	0	62
	WBR	9:00 am to 9:15 am	7	3	0	10
	EBT	9:00 am to 9:15 am	35	2	0	37
	EBL	9:00 am to 9:15 am	6	0	0	6
	NBT	9:00 am to 9:15 am	0	0	0	0
	NBR	9:00 am to 9:15 am	90	5	0	95
	NBL	9:00 am to 9:15 am	60	2	0	62
	WBT	9:15 am to 9:30 am	42	4	0	46
	WBR	9:15 am to 9:30 am	2	5	1	8
	EBT	9:15 am to 9:30 am	28	3	0	31
	EBL	9:15 am to 9:30 am	3	0	1	4
	NBT	9:15 am to 9:30 am	0	0	0	0
	NBR	9:15 am to 9:30 am	7	5	0	12
	NBL	9:15 am to 9:30 am	49	1	1	51
	WBT	9:30 am to 9:45 am	44	6	0	50
	WBR	9:30 am to 9:45 am	3	0	0	3
	EBT	9:30 am to 9:45 am	37	3	0	40
	EBL	9:30 am to 9:45 am	3	0	0	3
	NBT	9:30 am to 9:45 am	0	0	0	0
	NBR	9:30 am to 9:45 am	4	4	0	8
	NBL	9:30 am to 9:45 am	42	1	0	43
	WBT	9:45 am to 10:00 am	48	5	0	53
	WBR	9:45 am to 10:00 am	22	1	0	23
	EBT	9:45 am to 10:00 am	45	1	0	46
	EBL	9:45 am to 10:00 am	6	0	0	6
	NBT	9:45 am to 10:00 am	0	0	0	0
	NBR	9:45 am to 10:00 am	9	3	0	12
	NBL	9:45 am to 10:00 am	5	1	0	6

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
	WBT	4:00 pm to 4:15 pm	66	1	0	67
	WBR	4:00 pm to 4:15 pm	8	2	0	10
	EBT	4:00 pm to 4:15 pm	59	4	0	63
	EBL	4:00 pm to 4:15 pm	17	1	0	18
	NBT	4:00 pm to 4:15 pm	0	0	0	0
	NBR	4:00 pm to 4:15 pm	20	4	0	24
	NBL	4:00 pm tp 4:15 pm	49	2	0	51
	WBT	4:15 pm to 4:30 pm	58	5	0	63
	WBR	4:15 pm to 4:30 pm	4	0	0	4
	EBT	4:15 pm to 4:30 pm	70	2	0	72
	EBL	4:15 pm to 4:30 pm	22	0	0	22
	NBT	4:15 pm to 4:30 pm	0	0	0	0
	NBR	4:15 pm to 4:30 pm	22	3	0	25
	NBL	4:15 pm to 4:30 pm	85	1	0	86
	WBT	4:30 pm to 4 :45 pm	58	2	1	61
	WBR	4:30 pm to 4 :45 pm	5	3	0	8
	EBT	4:30 pm to 4 :45 pm	61	1	0	62
	EBL	4:30 pm to 4 :45 pm	21	0	0	21
	NBT	4:30 pm to 4 :45 pm	0	0	0	0
	NBR	4:30 pm to 4 :45 pm	15	0	0	15
	NBL	4:30 pm to 4 :45 pm	88	2	0	90
	WBT	4:45 pm to 5:00 pm	99	6	0	105
	WBR	4:45 pm to 5:00 pm	12	1	0	13
	EBT	4:45 pm to 5:00 pm	68	0	0	68
	EBL	4:45 pm to 5:00 pm	21	0	0	21
	NBT	4:45 pm to 5:00 pm	0	0	0	0
	NBR	4:45 pm to 5:00 pm	34	1	0	35
	NBL	4:45 pm to 5:00 pm	78	1	1	80
	WBT	5:00 pm to 5:15 pm	120	3	0	123
	WBR	5:00 pm to 5:15 pm	9	2	0	11
	EBT	5:00 pm to 5:15 pm	75	1	0	76
	EBL	5:00 pm to 5:15 pm	23	1	0	24
	NBT	5:00 pm to 5:15 pm	1	0	0	1
	NBR	5:00 pm to 5:15 pm	32	2	0	34
	NBL	5:00 pm to 5:15 pm	74	1	0	75
	WBT	5:15 pm to 5:30 pm	86	1	0	87
	WBR	5:15 pm to 5:30 pm	2	2	0	4
	EBT	5:15 pm to 5:30 pm	84	0	1	85
	EBL	5:15 pm to 5:30 pm	24	1	0	25
	NBT	5:15 pm to 5:30 pm	0	0	0	0
	NBR	5:15 pm to 5:30 pm	15	3	0	18
	NBL	5:15 pm to 5:30 pm	76	0	1	77
	WBT	5:30 pm to 5:45 pm	62	1	0	63
	WBR	5:30 pm to 5:45 pm	5	1	0	6
	EBT	5:30 pm to 5:45 pm	61	0	0	61
	EBL	5:30 pm to 5:45 pm	15	0	0	15
	NBT	5:30 pm to 5:45 pm	0	0	0	0
	NBR	5:30 pm to 5:45 pm	21	2	2	25
	NBL	5:30 pm to 5:45 pm	84	1	1	86
	WBT	5:45 pm to 6:00 pm	52	3	0	55
	WBR	5:45 pm to 6:00 pm	8	1	0	9
	EBT	5:45 pm to 6:00 pm	63	1	0	64
	EBL	5:45 pm to 6:00 pm	26	0	0	26
	NBT	5:45 pm to 6:00 pm	0	0	0	0
	NBR	5:45 pm to 6:00 pm	4	0	0	4
	NBL	5:45 pm to 6:00 pm	62	0	0	62

F(Freeway)

OFF (Off-Ramp)

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: UB	NBT - F	8:00 am to 8:15 am	69	11	2	82
	NBT - OFF	8:00 am to 8:15 am	85	6	3	94
	NBT - F	8:15 am to 8:30 am	82	10	2	94
	NBT - OFF	8:15 am to 8:30 am	82	4	0	86
	NBT - F	8:30 am to 8:45 am	53	8	4	65
	NBT - OFF	8:30 am to 8:45 am	73	5	0	78
	NBT - F	8:45 am to 9:00 am	71	12	4	87
	NBT - OFF	8:45 am to 9:00 am	73	3	3	79
	NBT - F	9:00 am to 9:15 am	72	14	2	88
	NBT - OFF	9:00 am to 9:15 am	49	6	1	56
	NBT - F	9:15 am to 9:30 am	49	14	4	67
	NBT - OFF	9:15 am to 9:30 am	56	6	0	62
	NBT - F	9:30 am to 9:45 am	66	16	2	84
	NBT - OFF	9:30 am to 9:45 am	45	4	1	50
	NBT - F	9:45 am to 10:00 am	57	16	2	75
	NBT - OFF	9:45 am to 10:00 am	76	6	0	82
	NBT - F	4:00 pm to 4:15 pm	86	12	0	98
	NBT - OFF	4:00 pm to 4:15 pm	89	6	0	95
	NBT - F	4:15 pm to 4:30 pm	95	3	0	98
	NBT - OFF	4:15 pm to 4:30 pm	115	3	0	118
	NBT - F	4:30 pm to 4:45 pm	90	5	0	95
	NBT - OFF	4:30 pm to 4:45 pm	95	2	0	97
	NBT - F	4:45 pm to 5:00 pm	86	4	0	90
	NBT - OFF	4:45 pm to 5:00 pm	115	1	1	117
	NBT - F	5:00 pm to 5:15 pm	106	3	0	109
	NBT - OFF	5:00 pm to 5:15 pm	109	3	0	112
	NBT - F	5:15 pm to 5:30 pm	96	2	0	98
	NBT - OFF	5:15 pm to 5:30 pm	95	3	0	98
	NBT - F	5:30 pm to 5:45 pm	65	4	0	69
	NBT - OFF	5:30 pm to 5:45 pm	91	3	0	94
	NBT - F	5:45 pm to 6:00 pm	63	7	0	70
	NBT - OFF	5:45 pm to 6:00 pm	67	0	0	67

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: KV	EBT	8:00 am to 8:15 am	46	1	0	47
	EBR	8:00 am to 8:15 am	50	1	0	51
	WBT	8:00 am to 8:15 am	116	5	1	122
	WBL	8:00 am to 8:15 am	13	3	0	16
	SBT	8:00 am to 8:15 am	0	0	0	0
	SBR	8:00 am to 8:15 am	9	0	0	9
	SBL	8:00 am to 8:15 am	7	0	0	7
	EBT	8:15 am to 8:30 am	19	5	0	24
	EBR	8:15 am to 8:30 am	62	0	1	63
	WBT	8:15 am to 8:30 am	108	1	0	109
	WBL	8:15 am to 8:30 am	32	3	0	35
	SBT	8:15 am to 8:30 am	0	0	0	0
	SBR	8:15 am to 8:30 am	3	0	0	3
	SBL	8:15 am to 8:30 am	3	1	0	4
	EBT	8:30 am to 8:45 am	33	1	0	34
	EBR	8:30 am to 8:45 am	46	3	0	49
	WBT	8:30 am to 8:45 am	113	2	0	115
	WBL	8:30 am to 8:45 am	7	2	0	9
	SBT	8:30 am to 8:45 am	0	0	0	0
	SBR	8:30 am to 8:45 am	8	0	0	8
	SBL	8:30 am to 8:45 am	2	4	1	7
	EBT	8:45 am to 9:00 am	37	1	0	38
	EBR	8:45 am to 9:00 am	38	0	0	38
	WBT	8:45 am to 9:00 am	110	0	1	111
	WBL	8:45 am to 9:00 am	8	4	0	12
	SBT	8:45 am to 9:00 am	0	0	0	0
	SBR	8:45 am to 9:00 am	9	0	0	9
	SBL	8:45 am to 9:00 am	6	2	0	8
	EBT	9:00 am to 9:15 am	33	1	0	34
	EBR	9:00 am to 9:15 am	38	1	2	41
	WBT	9:00 am to 9:15 am	85	2	1	88
	WBL	9:00 am to 9:15 am	8	2	2	12
	SBT	9:00 am to 9:15 am	0	0	0	0
	SBR	9:00 am to 9:15 am	8	0	0	8
	SBL	9:00 am to 9:15 am	3	3	0	6
	EBT	9:15 am to 9:30 am	29	0	1	30
	EBR	9:15 am to 9:30 am	52	0	0	52
	WBT	9:15 am to 9:30 am	87	3	0	90
	WBL	9:15 am to 9:30 am	7	4	0	11
	SBT	9:15 am to 9:30 am	0	0	0	0
	SBR	9:15 am to 9:30 am	5	0	1	6
	SBL	9:15 am to 9:30 am	2	0	0	2
	EBT	9:30 am to 9:45 am	37	1	0	38
	EBR	9:30 am to 9:45 am	53	2	1	56
	WBT	9:30 am to 9:45 am	70	2	0	72
	WBL	9:30 am to 9:45 am	9	3	0	12
	SBT	9:30 am to 9:45 am	0	0	0	0
	SBR	9:30 am to 9:45 am	9	1	0	10
	SBL	9:30 am to 9:45 am	4	2	0	6
	EBT	9:45 am to 10:00 am	48	2	0	50
	EBR	9:45 am to 10:00 am	51	3	0	54
	WBT	9:45 am to 10:00 am	96	1	1	98
	WBL	9:45 am to 10:00 am	9	5	0	14
	SBT	9:45 am to 10:00 am	0	0	0	0
	SBR	9:45 am to 10:00 am	11	0	0	11
	SBL	9:45 am to 10:00 am	4	1	0	5

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
	EBT	4:00 pm to 4:15 pm	81	3	0	84
	EBR	4:00 pm to 4:15 pm	81	0	0	81
	WBT	4:00 pm to 4:15 pm	132	2	0	134
	WBL	4:00 pm to 4:15 pm	12	1	0	13
	SBT	4:00 pm to 4:15 pm	0	0	0	0
	SBR	4:00 pm to 4:15 pm	4	1	0	5
	SBL	4:00 pm to 4:15 pm	5	5	0	10
	EBT	4:15 pm to 4:30 pm	90	2	0	92
	EBR	4:15 pm to 4:30 pm	74	2	0	76
	WBT	4:15 pm to 4:30 pm	139	4	0	143
	WBL	4:15 pm to 4:30 pm	11	1	0	12
	SBT	4:15 pm to 4:30 pm	0	0	0	0
	SBR	4:15 pm to 4:30 pm	8	1	1	10
	SBL	4:15 pm to 4:30 pm	5	0	0	5
	EBT	4:30 pm to 4:45 pm	76	0	0	76
	EBR	4:30 pm to 4:45 pm	85	1	0	86
	WBT	4:30 pm to 4:45 pm	136	2	1	139
	WBL	4:30 pm to 4:45 pm	11	2	0	13
	SBT	4:30 pm to 4:45 pm	1	0	0	1
	SBR	4:30 pm to 4:45 pm	13	0	0	13
	SBL	4:30 pm to 4:45 pm	6	1	0	7
	EBT	4:45 pm to 5:00 pm	83	0	0	83
	EBR	4:45 pm to 5:00 pm	76	0	0	76
	WBT	4:45 pm to 5:00 pm	150	1	0	151
	WBL	4:45 pm to 5:00 pm	18	5	0	23
	SBT	4:45 pm to 5:00 pm	0	0	0	0
	SBR	4:45 pm to 5:00 pm	7	1	0	8
	SBL	4:45 pm to 5:00 pm	8	0	0	8
	EBT	5:00 pm to 5:15 pm	92	1	0	93
	EBR	5:00 pm to 5:15 pm	106	0	0	106
	WBT	5:00 pm to 5:15 pm	165	1	1	167
	WBL	5:00 pm to 5:15 pm	39	3	0	42
	SBT	5:00 pm to 5:15 pm	0	0	0	0
	SBR	5:00 pm to 5:15 pm	16	0	0	16
	SBL	5:00 pm to 5:15 pm	7	1	0	8
	EBT	5:15 pm to 5:30 pm	99	2	1	102
	EBR	5:15 pm to 5:30 pm	82	1	0	83
	WBT	5:15 pm to 5:30 pm	143	0	0	143
	WBL	5:15 pm to 5:30 pm	24	1	0	25
	SBT	5:15 pm to 5:30 pm	0	0	0	0
	SBR	5:15 pm to 5:30 pm	8	0	0	8
	SBL	5:15 pm to 5:30 pm	4	0	0	4
	EBT	5:30 pm to 5:45 pm	70	0	0	70
	EBR	5:30 pm to 5:45 pm	75	0	0	75
	WBT	5:30 pm to 5:45 pm	136	1	0	137
	WBL	5:30 pm to 5:45 pm	15	1	0	16
	SBT	5:30 pm to 5:45 pm	0	0	0	0
	SBR	5:30 pm to 5:45 pm	12	0	0	12
	SBL	5:30 pm to 5:45 pm	10	1	0	11
	EBT	5:45 pm to 6:00 pm	68	0	0	68
	EBR	5:45 pm to 6:00 pm	66	1	0	67
	WBT	5:45 pm to 6:00 pm	105	0	0	105
	WBL	5:45 pm to 6:00 pm	10	3	0	13
	SBT	5:45 pm to 6:00 pm	0	0	0	0
	SBR	5:45 pm to 6:00 pm	10	0	0	10
	SBL	5:45 pm to 6:00 pm	5	0	0	5

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: VA	WBT	8:00 am to 8:15 am	125	1	2	128
	WBR	8:00 am to 8:15 am	9	2	0	11
	WBT	8:15 am to 8:30 am	112	0	0	112
	WBR	8:15 am to 8:30 am	6	0	0	6
	WBT	8:30 am to 8:45 am	99	1	0	100
	WBR	8:30 am to 8:45 am	16	0	1	17
	WBT	8:45 am to 9:00 am	104	1	2	107
	WBR	8:45 am to 9:00 am	12	0	0	12
	WBT	9:00 am to 9:15 am	88	2	0	90
	WBR	9:00 am to 9:15 am	5	0	0	5
	WBT	9:15 am to 9:30 am	85	3	2	90
	WBR	9:15 am to 9:30 am	9	0	0	9
	WBT	9:30 am to 9:45 am	65	2	1	68
	WBR	9:30 am to 9:45 am	9	0	0	9
	WBT	9:45 am to 10:00 am	98	2	0	100
	WBR	9:45 am to 10:00 am	16	0	0	16
Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
	WBT	4:00 pm to 4:15 pm	137	0	0	137
	WBR	4:00 pm to 4:15 pm	18	1	0	19
	WBT	4:15 pm to 4:30 pm	116	4	1	121
	WBR	4:15 pm to 4:30 pm	20	0	0	20
	WBT	4:30 pm to 4:45 pm	131	2	1	134
	WBR	4:30 pm to 4:45 pm	17	0	0	17
	WBT	4:45 pm to 5:00 pm	146	0	0	146
	WBR	4:45 pm to 5:00 pm	21	1	0	22
	WBT	5:00 pm to 5:15 pm	145	1	1	147
	WBR	5:00 pm to 5:15 pm	20	0	0	20
	WBT	5:15 pm to 5:30 pm	141	0	0	141
	WBR	5:15 pm to 5:30 pm	16	0	0	16
	WBT	5:30 pm to 5:45 pm	142	0	1	143
	WBR	5:30 pm to 5:45 pm	14	0	0	14
	WBT	5:45 pm to 6:00 pm	98	0	0	98
	WBR	5:45 pm to 6:00 pm	15	0	0	15

A.2.2 DI_{sig} at Lincoln

Figure A.11 Data collection device locations and corresponding traffic movements at the Lincoln site

Figure A.11 cont. Data collection device locations and corresponding traffic movements at the Lincoln site

Table A.11 Traffic volume counts at the Lincoln site

Date: 4/23/2024

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: TZ	EBT	8:00 am to 8:15 am	141	1	2	144
	EBL	8:00 am to 8:15 am	10	1	0	11
	EBT	8:15 am to 8:30 am	118	2	1	121
	EBL	8:15 am to 8:30 am	14	0	0	14
	EBT	8:30 am to 8:45 am	116	1	0	117
	EBL	8:30 am to 8:45 am	10	3	1	14
	EBT	8:45 am to 9:00 am	111	0	0	111
	EBL	8:45 am to 9:00 am	16	1	0	17
	EBT	9:00 am to 9:15 am	123	0	0	123
	EBL	9:00 am to 9:15 am	12	1	0	13
	EBT	9:15 am to 9:30 am	93	0	1	94
	EBL	9:15 am to 9:30 am	5	1	0	6
	EBT	9:30 am to 9:45 am	110	3	1	114
	EBL	9:30 am to 9:45 am	14	5	0	19
	EBT	9:45 am to 10:00 am	87	3	0	90
	EBL	9:45 am to 10:00 am	18	2	2	22
	EBT	4:00 pm to 4:15 pm	151	1	0	152
	EBL	4:00 pm to 4:15 pm	10	1	0	11
	EBT	4:15 pm to 4:30 pm	164	1	1	166
	EBL	4:15 pm to 4:30 pm	17	0	0	17
	EBT	4:30 pm to 4:45 pm	139	3	1	143
	EBL	4:30 pm to 4:45 pm	32	1	0	33
	EBT	4:45 pm to 5:00 pm	156	0	1	157
	EBL	4:45 pm to 5:00 pm	14	0	1	15
	EBT	5:00 pm to 5:15 pm	171	0	1	172
	EBL	5:00 pm to 5:15 pm	28	0	0	28
	EBT	5:15 pm to 5:30 pm	170	1	1	172
	EBL	5:15 pm to 5:30 pm	17	0	0	17
	EBT	5:30 pm to 5:45 pm	161	0	0	161
	EBL	5:30 pm to 5:45 pm	17	0	0	17
	EBT	5:45 pm to 6:00 pm	151	0	0	151
	EBL	5:45 pm to 6:00 pm	14	0	0	14

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: KV	WB	8:00 am to 8:15 am	145	5	1	151
	WB	8:15 am to 8:30 am	137	0	4	141
	WB	8:30 am to 8:45 am	149	6	0	155
	WB	8:45 am to 9:00 am	147	5	2	154
	WB	9:00 am to 9:15 am	113	3	3	119
	WB	9:15 am to 9:30 am	132	6	1	139
	WB	9:30 am to 9:45 am	126	3	2	131
	WB	9:45 am to 10:00 am	134	6	2	142
	WB	4:00 pm to 4:15 pm	232	1	2	235
	WB	4:15 pm to 4:30 pm	263	4	1	268
	WB	4:30 pm to 4:45 pm	257	1	0	258
	WB	4:45 pm to 5:00 pm	262	1	0	263
	WB	5:00 pm to 5:15 pm	288	0	0	288
	WB	5:15 pm to 5:30 pm	250	0	0	250
	WB	5:30 pm to 5:45 pm	190	1	0	191
	WB	5:45 pm to 6:00 pm	205	0	0	205
Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: VA	EB	8:00 am to 8:15 am	159	5	1	165
	EB	8:15 am to 8:30 am	152	2	1	155
	EB	8:30 am to 8:45 am	152	6	1	159
	EB	8:45 am to 9:00 am	135	4	0	139
	EB	9:00 am to 9:15 am	143	3	1	147
	EB	9:15 am to 9:30 am	128	3	1	132
	EB	9:30 am to 9:45 am	137	9	2	148
	EB	9:45 am to 10:00 am	127	7	7	141
	EB	4:00 pm to 4:15 pm	225	4	1	230
	EB	4:15 pm to 4:30 pm	199	6	1	206
	EB	4:30 pm to 4:45 pm	236	7	0	243
	EB	4:45 pm to 5:00 pm	189	0	2	191
	EB	5:00 pm to 5:15 pm	269	3	1	273
	EB	5:15 pm to 5:30 pm	225	2	1	228
	EB	5:30 pm to 5:45 pm	181	1	0	182
	EB	5:45 pm to 6:00 pm	151	0	0	151

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	EBR	8:00 am to 8:15 am	36	5	0	41
	EBT	8:00 am to 8:15 am	113	1	0	114
	SBR	8:00 am to 8:15 am	36	1	0	37
	SBT	8:00 am to 8:15 am	0	0	0	0
	SBL	8:00 am to 8:15 am	17	1	0	18
	EBR	8:15 am to 8:30 am	39	1	0	40
	EBT	8:15 am to 8:30 am	112	1	1	114
	SBR	8:15 am to 8:30 am	14	0	0	14
	SBT	8:15 am to 8:30 am	0	0	0	0
	SBL	8:15 am to 8:30 am	20	1	0	21
	EBR	8:30 am to 8:45 am	30	3	0	33
	EBT	8:30 am to 8:45 am	124	4	1	129
	SBR	8:30 am to 8:45 am	15	3	0	18
	SBT	8:30 am to 8:45 am	1	0	0	1
	SBL	8:30 am to 8:45 am	10	0	1	11
	EBR	8:45 am to 9:00 am	22	3	0	25
	EBT	8:45 am to 9:00 am	113	1	0	114
	SBR	8:45 am to 9:00 am	17	4	0	21
	SBT	8:45 am to 9:00 am	0	0	0	0
	SBL	8:45 am to 9:00 am	14	0	0	14
	EBR	9:00 am to 9:15 am	30	3	1	34
	EBT	9:00 am to 9:15 am	115	1	0	116
	SBR	9:00 am to 9:15 am	10	3	0	13
	SBT	9:00 am to 9:15 am	0	0	0	0
	SBL	9:00 am to 9:15 am	15	0	0	15
	EBR	9:15 am to 9:30 am	29	2	0	31
	EBT	9:15 am to 9:30 am	83	1	1	85
	SBR	9:15 am to 9:30 am	12	3	0	15
	SBT	9:15 am to 9:30 am	0	0	0	0
	SBL	9:15 am to 9:30 am	7	0	0	7
	EBR	9:30 am to 9:45 am	28	1	1	30
	EBT	9:30 am to 9:45 am	117	8	0	125
	SBR	9:30 am to 9:45 am	18	1	0	19
	SBT	9:30 am to 9:45 am	0	0	0	0
	SBL	9:30 am to 9:45 am	11	0	0	11
	EBR	9:45 am to 10:00 am	31	3	4	38
	EBT	9:45 am to 10:00 am	94	4	3	101
	SBR	9:45 am to 10:00 am	16	2	1	19
	SBT	9:45 am to 10:00 am	0	0	0	0
	SBL	9:45 am to 10:00 am	6	1	1	8

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2	EBR	4:00 pm to 4:15 pm	76	1	0	77
	EBT	4:00 pm to 4:15 pm	147	2	0	149
	SBR	4:00 pm to 4:15 pm	26	1	1	28
	SBT	4:00 pm to 4:15 pm	0	0	0	0
	SBL	4:00 pm to 4:15 pm	25	0	0	25
	EBR	4:15 pm to 4:30 pm	62	4	0	66
	EBT	4:15 pm to 4:30 pm	145	1	1	147
	SBR	4:15 pm to 4:30 pm	50	0	0	50
	SBT	4:15 pm to 4:30 pm	0	0	0	0
	SBL	4:15 pm to 4:30 pm	23	1	0	24
	EBR	4:30 pm to 4:45 pm	75	2	0	77
	EBT	4:30 pm to 4:45 pm	152	2	1	155
	SBR	4:30 pm to 4:45 pm	25	0	1	26
	SBT	4:30 pm to 4:45 pm	0	0	0	0
	SBL	4:30 pm to 4:45 pm	23	0	0	23
	EBR	4:45 pm to 5:00 pm	45	0	0	45
	EBT	4:45 pm to 5:00 pm	150	0	2	152
	SBR	4:45 pm to 5:00 pm	18	4	0	22
	SBT	4:45 pm to 5:00 pm	0	0	0	0
	SBL	4:45 pm to 5:00 pm	24	0	0	24
	EBR	5:00 pm to 5:15 pm	90	3	0	93
	EBT	5:00 pm to 5:15 pm	179	0	0	179
	SBR	5:00 pm to 5:15 pm	15	0	0	15
	SBT	5:00 pm to 5:15 pm	0	0	0	0
	SBL	5:00 pm to 5:15 pm	21	0	1	22
	EBR	5:15 pm to 5:30 pm	53	1	0	54
	EBT	5:15 pm to 5:30 pm	167	1	1	169
	SBR	5:15 pm to 5:30 pm	19	0	0	19
	SBT	5:15 pm to 5:30 pm	0	0	0	0
	SBL	5:15 pm to 5:30 pm	21	0	0	21
	EBR	5:30 pm to 5:45 pm	38	1	0	39
	EBT	5:30 pm to 5:45 pm	145	0	0	145
	SBR	5:30 pm to 5:45 pm	18	1	1	20
	SBT	5:30 pm to 5:45 pm	0	0	0	0
	SBL	5:30 pm to 5:45 pm	28	0	0	28
	EBR	5:45 pm to 6:00 pm	29	0	0	29
	EBT	5:45 pm to 6:00 pm	129	0	0	129
	SBR	5:45 pm to 6:00 pm	11	0	0	11
	SBT	5:45 pm to 6:00 pm	0	0	0	0
	SBL	5:45 pm to 6:00 pm	40	0	0	40

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: T2_UB	WBT	8:00 am to 8:15 am	135	6	0	141
	WBL	8:00 am to 8:15 am	42	0	1	43
	WBT	8:15 am to 8:30 am	115	6	2	123
	WBL	8:15 am to 8:30 am	40	1	0	41
	WBT	8:30 am to 8:45 am	115	7	1	123
	WBL	8:30 am to 8:45 am	50	2	0	52
	WBT	8:45 am to 9:00 am	125	6	2	133
	WBL	8:45 am to 9:00 am	46	1	0	47
	WBT	9:00 am to 9:15 am	100	5	2	107
	WBL	9:00 am to 9:15 am	27	1	1	29
	WBT	9:15 am to 9:30 am	110	6	2	118
	WBL	9:15 am to 9:30 am	35	0	0	35
	WBT	9:30 am to 9:45 am	116	9	2	127
	WBL	9:30 am to 9:45 am	33	0	0	33
	WBT	9:45 am to 10:00 am	126	6	2	134
	WBL	9:45 am to 10:00 am	22	1	0	23
	WBT	4:00 pm to 4:15 pm	171	4	0	175
	WBL	4:00 pm to 4:15 pm	68	0	1	69
	WBT	4:15 pm to 4:30 pm	163	3	1	167
	WBL	4:15 pm to 4:30 pm	98	2	0	100
	WBT	4:30 pm to 4:45 pm	185	1	0	186
	WBL	4:30 pm to 4:45 pm	78	0	0	78
	WBT	4:45 pm to 5:00 pm	193	0	0	193
	WBL	4:45 pm to 5:00 pm	76	0	0	76
	WBT	5:00 pm to 5:15 pm	187	2	0	189
	WBL	5:00 pm to 5:15 pm	103	0	0	103
	WBT	5:15 pm to 5:30 pm	183	1	0	184
	WBL	5:15 pm to 5:30 pm	80	0	0	80
	WBT	5:30 pm to 5:45 pm	141	2	0	143
	WBL	5:30 pm to 5:45 pm	57	0	0	57
	WBT	5:45 pm to 6:00 pm	167	0	0	167
	WBL	5:45 pm to 6:00 pm	69	0	0	69

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: K6	WBR	8:00 am to 8:15 am	22	0	0	22
	WBT	8:00 am to 8:15 am	120	5	0	125
	NBR	8:00 am to 8:15 am	92	1	0	93
	NBT	8:00 am to 8:15 am	1	0	0	1
	NBL	8:00 am to 8:15 am	55	3	0	58
	WBR	8:15 am to 8:30 am	28	0	1	29
	WBT	8:15 am to 8:30 am	116	2	1	119
	NBR	8:15 am to 8:30 am	58	1	1	60
	NBT	8:15 am to 8:30 am	0	0	0	0
	NBL	8:15 am to 8:30 am	41	4	1	46
	WBR	8:30 am to 8:45 am	22	1	1	24
	WBT	8:30 am to 8:45 am	122	5	1	128
	NBR	8:30 am to 8:45 am	65	0	0	65
	NBT	8:30 am to 8:45 am	0	0	0	0
	NBL	8:30 am to 8:45 am	43	4	0	47
	WBR	8:45 am to 9:00 am	23	0	0	23
	WBT	8:45 am to 9:00 am	125	5	1	131
	NBR	8:45 am to 9:00 am	54	0	0	54
	NBT	8:45 am to 9:00 am	0	0	0	0
	NBL	8:45 am to 9:00 am	45	2	1	48
	WBR	9:00 am to 9:15 am	16	0	1	17
	WBT	9:00 am to 9:15 am	93	4	4	101
	NBR	9:00 am to 9:15 am	43	0	0	43
	NBT	9:00 am to 9:15 am	0	0	0	0
	NBL	9:00 am to 9:15 am	31	3	0	34
	WBR	9:15 am to 9:30 am	16	0	0	16
	WBT	9:15 am to 9:30 am	115	6	1	122
	NBR	9:15 am to 9:30 am	37	1	0	38
	NBT	9:15 am to 9:30 am	0	0	0	0
	NBL	9:15 am to 9:30 am	35	3	0	38
	WBR	9:30 am to 9:45 am	12	0	1	13
	WBT	9:30 am to 9:45 am	118	3	1	122
	NBR	9:30 am to 9:45 am	32	0	0	32
	NBT	9:30 am to 9:45 am	1	0	0	1
	NBL	9:30 am to 9:45 am	30	5	1	36
	WBR	9:45 am to 10:00 am	20	2	0	22
	WBT	9:45 am to 10:00 am	108	4	2	114
	NBR	9:45 am to 10:00 am	44	2	0	46
	NBT	9:45 am to 10:00 am	0	0	0	0
	NBL	9:45 am to 10:00 am	39	3	0	42

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	T	Total
Miovision: K6	WBR	4:00 pm to 4:15 pm	28	0	0	28
	WBT	4:00 pm to 4:15 pm	204	1	1	206
	NBR	4:00 pm to 4:15 pm	84	1	2	87
	NBT	4:00 pm to 4:15 pm	1	0	0	1
	NBL	4:00 pm to 4:15 pm	40	2	0	42
	WBR	4:15 pm to 4:30 pm	28	0	0	28
	WBT	4:15 pm to 4:30 pm	230	4	0	234
	NBR	4:15 pm to 4:30 pm	77	0	0	77
	NBT	4:15 pm to 4:30 pm	1	0	0	1
	NBL	4:15 pm to 4:30 pm	31	1	1	33
	WBR	4:30 pm to 4:45 pm	28	0	0	28
	WBT	4:30 pm to 4:45 pm	234	1	0	235
	NBR	4:30 pm to 4:45 pm	79	0	0	79
	NBT	4:30 pm to 4:45 pm	0	0	0	0
	NBL	4:30 pm to 4:45 pm	31	0	0	31
	WBR	4:45 pm to 5:00 pm	30	1	0	31
	WBT	4:45 pm to 5:00 pm	228	0	0	228
	NBR	4:45 pm to 5:00 pm	79	0	0	79
	NBT	4:45 pm to 5:00 pm	0	0	0	0
	NBL	4:45 pm to 5:00 pm	35	0	0	35
	WBR	5:00 pm to 5:15 pm	30	0	0	30
	WBT	5:00 pm to 5:15 pm	255	0	0	255
	NBR	5:00 pm to 5:15 pm	89	1	0	90
	NBT	5:00 pm to 5:15 pm	1	0	0	1
	NBL	5:00 pm to 5:15 pm	37	2	0	39
	WBR	5:15 pm to 5:30 pm	30	0	0	30
	WBT	5:15 pm to 5:30 pm	230	0	0	230
	NBR	5:15 pm to 5:30 pm	111	0	1	112
	NBT	5:15 pm to 5:30 pm	1	0	0	1
	NBL	5:15 pm to 5:30 pm	33	1	0	34
	WBR	5:30 pm to 5:45 pm	25	0	0	25
	WBT	5:30 pm to 5:45 pm	168	1	0	169
	NBR	5:30 pm to 5:45 pm	120	0	0	120
	NBT	5:30 pm to 5:45 pm	0	0	0	0
	NBL	5:30 pm to 5:45 pm	35	1	0	36
	WBR	5:45 pm to 6:00 pm	15	0	0	15
	WBT	5:45 pm to 6:00 pm	194	0	0	194
	NBR	5:45 pm to 6:00 pm	116	0	0	116
	NBT	5:45 pm to 6:00 pm	1	0	0	1
	NBL	5:45 pm to 6:00 pm	37	0	0	37

A.2.3 DDI at Lincoln

Figure A.12 Data collection device locations and corresponding traffic movements at the Lincoln site

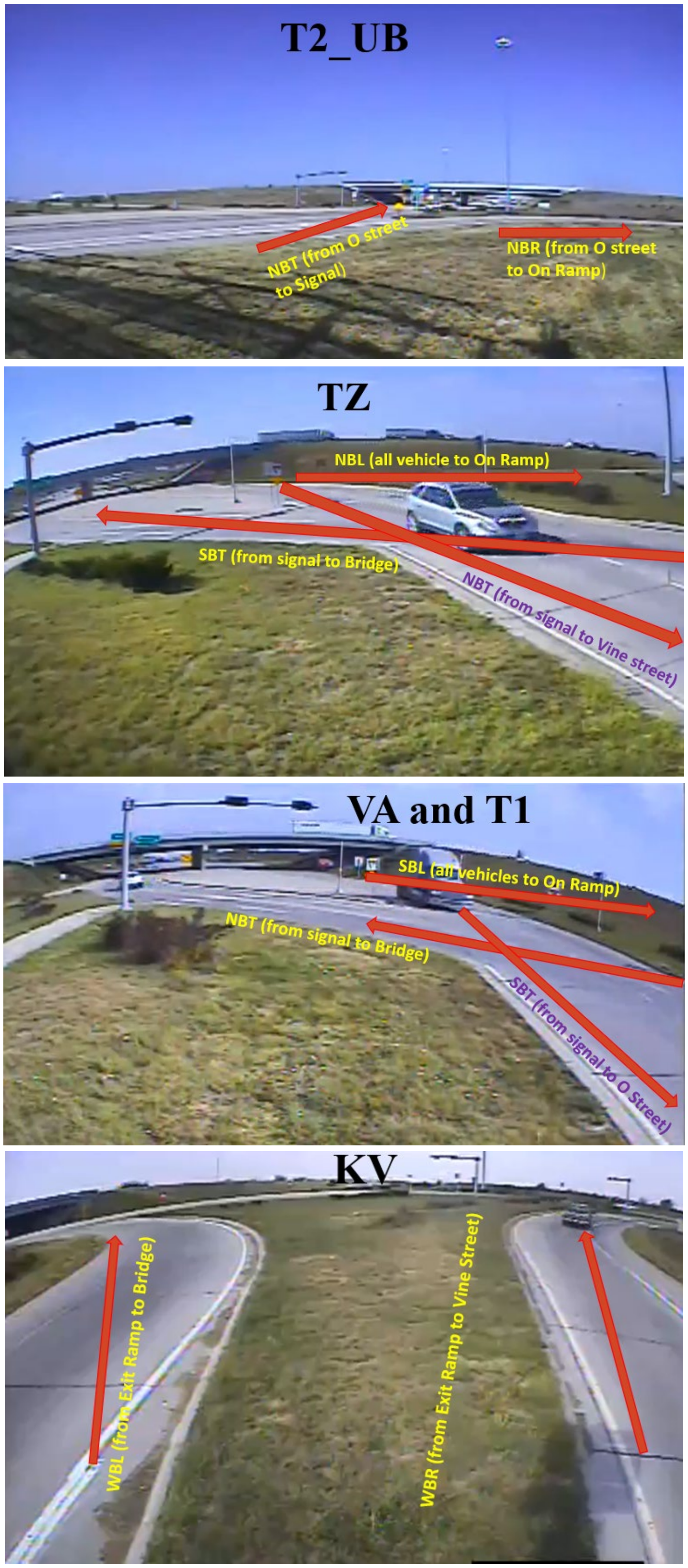


Figure A.12 cont. Data collection device locations and corresponding traffic movements at the Lincoln site

Table A.12 traffic volume counts

Date: 9/19/2023

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	SUT	Total
Miovision: TZ	SBT	8:00 am to 8:15 am	189	1	1	191
	NBT	8:00 am to 8:15 am	38	1	0	39
	NBL	8:00 am to 8:15 am	12	1	1	14
	SBT	8:15 am to 8:30 am	169	3	0	172
	NBT	8:15 am to 8:30 am	38	1	0	39
	NBL	8:15 am to 8:30 am	8	2	0	10
	SBT	8:30 am to 8:45 am	167	4	1	172
	NBT	8:30 am to 8:45 am	33	0	0	33
	NBL	8:30 am to 8:45 am	16	6	0	22
	SBT	8:45 am to 9:00 am	148	2	0	150
	NBT	8:45 am to 9:00 am	40	2	2	44
	NBL	8:45 am to 9:00 am	13	4	1	18
	SBT	9:00 am to 9:15 am	132	1	1	134
	NBT	9:00 am to 9:15 am	30	0	0	30
	NBL	9:00 am to 9:15 am	13	6	1	20
	SBT	9:15 am to 9:30 am	101	1	0	102
	NBT	9:15 am to 9:30 am	34	2	1	37
	NBL	9:15 am to 9:30 am	11	5	4	20
	SBT	9:30 am to 9:45 am	92	4	1	97
	NBT	9:30 am to 9:45 am	41	2	0	43
	NBL	9:30 am to 9:45 am	12	1	0	13
	SBT	9:45 am to 10:00 am	87	0	1	88
	NBT	9:45 am to 10:00 am	29	1	0	30
	NBL	9:45 am to 10:00 am	10	7	1	18
	SBT	4:00 pm to 4:15 pm	174	1	0	175
	NBT	4:00 pm to 4:15 pm	88	0	1	89
	NBL	4:00 pm to 4:15 pm	20	5	0	25
	SBT	4:15 pm to 4:30 pm	135	0	2	137
	NBT	4:15 pm to 4:30 pm	86	0	0	86
	NBL	4:15 pm to 4:30 pm	16	2	0	18
	SBT	4:30 pm to 4:45 pm	200	1	0	201
	NBT	4:30 pm to 4:45 pm	100	1	1	102
	NBL	4:30 pm to 4:45 pm	16	3	1	20
	SBT	4:45 pm to 5:00 pm	187	4	0	191
	NBT	4:45 pm to 5:00 pm	93	1	0	94
	NBL	4:45 pm to 5:00 pm	16	2	0	18
	SBT	5:00 pm to 5:15 pm	182	3	0	185
	NBT	5:00 pm to 5:15 pm	106	1	0	107
	NBL	5:00 pm to 5:15 pm	18	0	0	18
	SBT	5:15 pm to 5:30 pm	200	1	0	201
	NBT	5:15 pm to 5:30 pm	136	0	0	136
	NBL	5:15 pm to 5:30 pm	11	6	0	17
	SBT	5:30 pm to 5:45 pm	216	1	0	217
	NBT	5:30 pm to 5:45 pm	94	0	0	94
	NBL	5:30 pm to 5:45 pm	18	4	0	22
	SBT	5:45 pm to 6:00 pm	151	2	0	153
	NBT	5:45 pm to 6:00 pm	76	1	0	77
	NBL	5:45 pm to 6:00 pm	5	2	0	7

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	SUT	Total
Miovision: KV	WBL	8:00 am to 8:15 am	15	7	2	24
	WBR	8:00 am to 8:15 am	81	7	0	88
	WBL	8:15 am to 8:30 am	12	4	0	16
	WBR	8:15 am to 8:30 am	67	2	1	70
	WBL	8:30 am to 8:45 am	10	4	1	15
	WBR	8:30 am to 8:45 am	44	1	0	45
	WBL	8:45 am to 9:00 am	7	5	0	12
	WBR	8:45 am to 9:00 am	49	0	1	50
	WBL	9:00 am to 9:15 am	8	5	1	14
	WBR	9:00 am to 9:15 am	35	1	0	36
	WBL	9:15 am to 9:30 am	6	8	0	14
	WBR	9:15 am to 9:30 am	34	1	1	36
	WBL	9:30 am to 9:45 am	6	5	0	11
	WBR	9:30 am to 9:45 am	23	2	0	25
	WBL	9:45 am to 10:00 am	7	4	0	11
	WBR	9:45 am to 10:00 am	28	1	0	29
	WBL	4:00 pm to 4:15 pm	25	6	0	31
	WBR	4:00 pm to 4:15 pm	135	2	0	137
	WBL	4:15 pm to 4:30 pm	15	4	0	19
	WBR	4:15 pm to 4:30 pm	127	1	0	128
	WBL	4:30 pm to 4:45 pm	22	3	0	25
	WBR	4:30 pm to 4:45 pm	140	0	1	141
	WBL	4:45 pm to 5:00 pm	23	4	0	27
	WBR	4:45 pm to 5:00 pm	159	1	0	160
	WBL	5:00 pm to 5:15 pm	16	3	0	19
	WBR	5:00 pm to 5:15 pm	182	1	0	183
	WBL	5:15 pm to 5:30 pm	16	1	0	17
	WBR	5:15 pm to 5:30 pm	201	1	0	202
	WBL	5:30 pm to 5:45 pm	18	7	1	26
	WBR	5:30 pm to 5:45 pm	183	0	0	183
	WBL	5:45 pm to 6:00 pm	14	4	1	19
	WBR	5:45 pm to 6:00 pm	116	0	0	116

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	SUT	Total
Miovision: VA	NBT	8:00 am to 8:15 am	51	1	0	52
	SBT	8:00 am to 8:15 am	93	7	3	103
	SBL	8:00 am to 8:15 am	116	0	2	118
	NBT	8:15 am to 8:30 am	45	2	0	47
	SBT	8:15 am to 8:30 am	75	7	0	82
	SBL	8:15 am to 8:30 am	106	1	0	107
	NBT	8:30 am to 8:45 am	43	6	0	49
	SBT	8:30 am to 8:45 am	75	5	2	82
	SBL	8:30 am to 8:45 am	106	2	1	109
	NBT	8:45 am to 9:00 am	51	6	3	60
	SBT	8:45 am to 9:00 am	57	3	0	60
	SBL	8:45 am to 9:00 am	96	3	0	99
	NBT	9:00 am to 9:15 am	43	3	0	46
	SBT	9:00 am to 9:15 am	71	5	2	78
	SBL	9:00 am to 9:15 am	64	1	1	66
	NBT	9:15 am to 9:30 am	40	7	2	49
	SBT	9:15 am to 9:30 am	52	10	0	62
	SBL	9:15 am to 9:30 am	57	1	0	58
	NBT	9:30 am to 9:45 am	51	3	0	54
	SBT	9:30 am to 9:45 am	45	5	1	51
	SBL	9:30 am to 9:45 am	50	2	1	53
	NBT	9:45 am to 10:00 am	37	8	1	46
	SBT	9:45 am to 10:00 am	34	5	3	42
	SBL	9:45 am to 10:00 am	55	0	1	56
	NBT	4:00 pm to 4:15 pm	99	5	0	104
	SBT	4:00 pm to 4:15 pm	87	7	0	94
	SBL	4:00 pm to 4:15 pm	115	0	0	115
	NBT	4:15 pm to 4:30 pm	91	2	0	93
	SBT	4:15 pm to 4:30 pm	64	4	0	68
	SBL	4:15 pm to 4:30 pm	85	0	1	86
	NBT	4:30 pm to 4:45 pm	105	2	1	108
	SBT	4:30 pm to 4:45 pm	93	2	0	95
	SBL	4:30 pm to 4:45 pm	118	1	0	119
	NBT	4:45 pm to 5:00 pm	102	2	0	104
	SBT	4:45 pm to 5:00 pm	97	3	0	100
	SBL	4:45 pm to 5:00 pm	116	4	0	120
	NBT	5:00 pm to 5:15 pm	113	0	0	113
	SBT	5:00 pm to 5:15 pm	70	6	0	76
	SBL	5:00 pm to 5:15 pm	122	1	0	123
	NBT	5:15 pm to 5:30 pm	133	6	0	139
	SBT	5:15 pm to 5:30 pm	99	2	0	101
	SBL	5:15 pm to 5:30 pm	121	1	0	122
	NBT	5:30 pm to 5:45 pm	104	4	0	108
	SBT	5:30 pm to 5:45 pm	83	7	0	90
	SBL	5:30 pm to 5:45 pm	148	0	0	148
	NBT	5:45 pm to 6:00 pm	77	4	0	81
	SBT	5:45 pm to 6:00 pm	64	6	1	71
	SBL	5:45 pm to 6:00 pm	102	2	0	104

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	SUT	Total
Miovision: T2	EBL	8:00 am to 8:15 am	2	0	0	2
	EBR	8:00 am to 8:15 am	19	3	1	23
	EBL	8:15 am to 8:30 am	2	1	0	3
	EBR	8:15 am to 8:30 am	18	0	0	18
	EBL	8:30 am to 8:45 am	2	0	0	2
	EBR	8:30 am to 8:45 am	11	2	0	13
	EBL	8:45 am to 9:00 am	4	0	0	4
	EBR	8:45 am to 9:00 am	14	0	0	14
	EBL	9:00 am to 9:15 am	0	0	0	0
	EBR	9:00 am to 9:15 am	11	2	0	13
	EBL	9:15 am to 9:30 am	2	1	1	4
	EBR	9:15 am to 9:30 am	14	2	0	16
	EBL	9:30 am to 9:45 am	0	0	0	0
	EBR	9:30 am to 9:45 am	13	3	0	16
	EBL	9:45 am to 10:00 am	1	0	0	1
	EBR	9:45 am to 10:00 am	9	10	0	19
	EBL	4:00 pm to 4:15 pm	6	0	0	6
	EBR	4:00 pm to 4:15 pm	15	3	0	18
	EBL	4:15 pm to 4:30 pm	9	0	0	9
	EBR	4:15 pm to 4:30 pm	11	1	0	12
	EBL	4:30 pm to 4:45 pm	11	0	0	11
	EBR	4:30 pm to 4:45 pm	18	3	1	22
	EBL	4:45 pm to 5:00 pm	4	1	0	5
	EBR	4:45 pm to 5:00 pm	20	5	0	25
	EBL	5:00 pm to 5:15 pm	7	1	0	8
	EBR	5:00 pm to 5:15 pm	14	7	0	21
	EBL	5:15 pm to 5:30 pm	7	0	0	7
	EBR	5:15 pm to 5:30 pm	21	5	0	26
	EBL	5:30 pm to 5:45 pm	7	0	0	7
	EBR	5:30 pm to 5:45 pm	18	7	0	25
	EBL	5:45 pm to 6:00 pm	4	0	0	4
	EBR	5:45 pm to 6:00 pm	21	6	0	27

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	SUT	Total
Miovision: T2_UB	NBT	8:00 am to 8:15 am	47	1	1	49
	NBR	8:00 am to 8:15 am	7	5	0	12
	NBT	8:15 am to 8:30 am	47	2	0	49
	NBR	8:15 am to 8:30 am	4	7	2	13
	NBT	8:30 am to 8:45 am	47	6	1	54
	NBR	8:30 am to 8:45 am	17	3	0	20
	NBT	8:45 am to 9:00 am	52	7	2	61
	NBR	8:45 am to 9:00 am	6	3	2	11
	NBT	9:00 am to 9:15 am	43	4	0	47
	NBR	9:00 am to 9:15 am	5	0	0	5
	NBT	9:15 am to 9:30 am	39	7	2	48
	NBR	9:15 am to 9:30 am	9	7	0	16
	NBT	9:30 am to 9:45 am	50	3	0	53
	NBR	9:30 am to 9:45 am	8	9	0	17
	NBT	9:45 am to 10:00 am	38	8	1	47
	NBR	9:45 am to 10:00 am	10	4	0	14
	NBT	4:00 pm to 4:15 pm	100	5	1	106
	NBR	4:00 pm to 4:15 pm	45	8	2	55
	NBT	4:15 pm to 4:30 pm	95	2	0	97
	NBR	4:15 pm to 4:30 pm	21	7	0	28
	NBT	4:30 pm to 4:45 pm	105	3	1	109
	NBR	4:30 pm to 4:45 pm	20	1	0	21
	NBT	4:45 pm to 5:00 pm	107	2	0	109
	NBR	4:45 pm to 5:00 pm	14	1	0	15
	NBT	5:00 pm to 5:15 pm	115	0	0	115
	NBR	5:00 pm to 5:15 pm	27	2	0	29
	NBT	5:15 pm to 5:30 pm	141	6	0	147
	NBR	5:15 pm to 5:30 pm	17	1	0	18
	NBT	5:30 pm to 5:45 pm	104	4	0	108
	NBR	5:30 pm to 5:45 pm	9	1	1	11
	NBT	5:45 pm to 6:00 pm	78	3	0	81
	NBR	5:45 pm to 6:00 pm	19	2	0	21

Recording Device	Movements	Time	Vehicle Classification			
			PC	TT	SUT	Total
Miovision: K6	SBT	8:00 am to 8:15 am	193	0	3	196
	SBR	8:00 am to 8:15 am	8	0	0	8
	SBT	8:15 am to 8:30 am	163	3	1	167
	SBR	8:15 am to 8:30 am	5	0	0	5
	SBT	8:30 am to 8:45 am	169	4	1	174
	SBR	8:30 am to 8:45 am	1	0	0	1
	SBT	8:45 am to 9:00 am	151	2	0	153
	SBR	8:45 am to 9:00 am	3	0	0	3
	SBT	9:00 am to 9:15 am	126	1	5	132
	SBR	9:00 am to 9:15 am	5	0	0	5
	SBT	9:15 am to 9:30 am	90	0	2	92
	SBR	9:15 am to 9:30 am	0	0	0	0
	SBT	9:30 am to 9:45 am	91	4	2	97
	SBR	9:30 am to 9:45 am	4	0	1	5
	SBT	9:45 am to 10:00 am	84	0	3	87
	SBR	9:45 am to 10:00 am	1	0	0	1
	SBT	4:00 pm to 4:15 pm	173	1	0	174
	SBR	4:00 pm to 4:15 pm	4	0	0	4
	SBT	4:15 pm to 4:30 pm	138	0	1	139
	SBR	4:15 pm to 4:30 pm	4	0	0	4
	SBT	4:30 pm to 4:45 pm	203	2	0	205
	SBR	4:30 pm to 4:45 pm	4	0	0	4
	SBT	4:45 pm to 5:00 pm	183	3	0	186
	SBR	4:45 pm to 5:00 pm	4	0	1	5
	SBT	5:00 pm to 5:15 pm	186	3	0	189
	SBR	5:00 pm to 5:15 pm	5	0	0	5
	SBT	5:15 pm to 5:30 pm	199	1	0	200
	SBR	5:15 pm to 5:30 pm	9	0	0	9
	SBT	5:30 pm to 5:45 pm	180	1	0	181
	SBR	5:30 pm to 5:45 pm	5	0	0	5
	SBT	5:45 pm to 6:00 pm	142	3	0	145
	SBR	5:45 pm to 6:00 pm	4	0	0	4

Appendix B LOS Thresholds

The LOS thresholds for TWSC intersections and roundabouts are based on Exhibit 22-8, for RCUT and signalized intersections on Exhibit 23-13, and for interchanges on Exhibit 23-10 of the Highway Capacity Manual, 2016 edition. The LOS threshold values for different types of intersections and interchanges are summarized in the table below.

LOS	**Control Delays (seconds/vehicle)**		
	TWSC Intersections and Roundabouts	**Signalized Intersections and RCUTs**	**Interchanges**
A	≤ 10	≤ 10	≤ 15
B	> 10-15	> 10-20	> 15-30
C	> 15-25	> 20-35	> 30-55
D	> 25-35	> 35-55	> 55-85
E	> 35-50	> 55-80	> 85-120
F	> 50	> 80	> 120

Appendix C Lists of TWSC Intersections at Rural Expressways

Int_ID	County	Geographical Coordinates (Latitude, Longitude)	Major Approach	Minor Approach
87	Saunders	41.147609, -96.630847	US-77	Hwy 66
88	Saunders	41.060021, -96.640270	US-77	Main street
89	Lancaster	40.929193, -96.643494	US-77	Waverly Rd
90	Lancaster	40.625038, -96.705717	US-77	Hickman Rd
91	Gage	40.537549, -96.708542	US-77	W Hallam Rd/Firth Rd
92	Platte	41.337160, -97.368235	US-81	41 Rd/141st Rd
93	Madison	41.828707, -97.443630	US-81	E 2nd St/828th Rd
94	Madison	41.902465, -97.426454	US-81	833rd Rd
95	Pierce	42.105359, -97.426601	US-81	847th Rd
96	Thayer	40.016801, -97.612230	US-81	Hwy 8
97	Thayer	40.147215, -97.570777	US-81	Hwy 136
98	Fillmore	40.422905, -97.596251	US-81	Hwy 74
99	Fillmore	40.538854, -97.586935	US-81	R St
100	York	40.742155, -97.605045	US-81	Rd 4/W O St
101	York	40.814437, -97.597670	US-81	S 50th St
102	York	40.865379, -97.619377	US-81	Recharge Rd/W 4th St
103	York	40.885875, -97.615732	US-81	I-80 Alt/W 25th St
104	York	40.901935, -97.599418	US-81	Rd 15/I-80 Alt
105	Madison	42.025328, -97.601217	US-275	546th Avenue
106	Dakota	42.429895, -96.434366	US-75	164th St
107	Scotts Bluff	41.784807, -103.674557	US-71	Sandberg Rd
108	Sarpy	41.076749, -95.891120	US-34	Harlan Lewis Rd

Appendix D Crash Frequency of TWSC Intersection using Negative Binomial Model

The table below shows crash frequency of TWSC based on the negative binomial Model applied to all AADT scenarios.

AADT (Major Approach)	**AADT (Minor Approach)**	**Intersection AADT***	**5 Years Crash Frequency**	**Per Year Crash Frequency***
1,500	5,00	2,000	6.01	1.20
1,500	2,500	4,000	7.08	1.42
1,500	5,000	6,500	8.69	1.74
1,500	7,500	9,000	10.66	2.13
2,500	5,00	3,000	6.52	1.30
2,500	2,500	5,000	7.68	1.54
2,500	5,000	7,500	9.43	1.89
2,500	7,500	10,000	11.57	2.31
5,000	5,00	5,500	8.01	1.60
5,000	2,500	7,500	9.43	1.89
5,000	5,000	10,000	11.57	2.31
5,000	7,500	12,500	14.20	2.84
10,000	5,00	10,500	12.05	2.41
10,000	2,500	12,500	14.20	2.84
10,000	5,000	15,000	17.42	3.48
10,000	7,500	17,500	21.38	4.28
15,000	5,00	15,500	18.15	3.63
15,000	2,500	17,500	21.38	4.28
15,000	5,000	20,000	26.23	5.25
15,000	7,500	22,500	32.19	6.44
20,000	5,00	20,500	27.33	5.47
20,000	2,500	22,500	32.19	6.44
20,000	5,000	25,000	39.50	7.90
20,000	7,500	27,500	48.46	9.69
25,000	5,00	25,500	41.15	8.23
25,000	2,500	27,500	48.46	9.69
25,000	5,000	30,000	59.47	11.89
25,000	7,500	32,500	72.97	14.59

Note: *underlined AADT values are outside of negative binomial model range

Appendix E Crash Frequency of TWSC and Signalized Intersections using HSM SPFs

A table of crash frequency (per year) for four-legged TWSC intersections (base case) and four-legged signalized intersections in multi-lane highways and associated factors is shown below.

AADT (Major Approach)	AADT (Minor Approach)	Crash Frequency TWSC Intersection (all crashes)	Crash Frequency Signalized Intersection (all crashes)*	Factors**
1,500	5,00	0.36	1.21	3.37
1,500	2,500	0.74	2.09	2.82
1,500	5,000	1.01	2.64	2.61
1,500	7,500	1.21	3.02	2.50
2,500	5,00	0.55	1.76	3.16
2,500	2,500	1.14	3.02	2.65
2,500	5,000	1.56	3.81	2.45
2,500	7,500	1.87	4.37	2.34
5,000	5,00	1.00	2.90	2.90
5,000	2,500	2.05	4.98	2.43
5,000	5,000	2.80	6.29	2.25
5,000	7,500	3.36	7.21	2.15
10,000	5,00	1.80	4.78	2.66
10,000	2,500	3.70	8.22	2.22
10,000	5,000	5.04	10.38	2.06
10,000	7,500	6.05	11.90	1.97
15,000	5,00	2.54	6.40	2.53
15,000	2,500	5.21	11.01	2.11
15,000	5,000	7.11	13.91	1.96
15,000	7,500	8.53	15.95	1.87
20,000	5,00	3.24	7.88	2.44
20,000	2,500	6.65	13.56	2.04
20,000	5,000	9.08	17.13	1.89
20,000	7,500	10.89	19.63	1.80
25,000	5,00	3.91	9.26	2.37
25,000	2,500	8.04	15.93	1.98
25,000	5,000	10.97	20.12	1.83
25,000	7,500	13.15	23.07	1.75
			Average	**2.33**

*using co-efficient values of -7.182 for a, 0.7222 for b and 0.337 for c in Equation (5.6)
**Factor: crash frequency of signalized intersection divided by TWSC intersection