EVALUATING THE PERFORMANCE AND SAFETY EFFECTIVENESS OF ROUNDABOUTS- AN UPDATE

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16. Abstract
The objective of this research was to evaluate driver behavior, safety benefits, operational and environmental benefits of roundabouts in Michigan. To that end, a list of all 180 known roundabouts was compiled from various sources. For comparison purposes, a group of reference intersections was also compiled that had similar traffic volume, roadway characteristics, and traffic control prior to roundabout construction. Field data collection was carried out at 18 sites that included collection of driver speed selection behavior, gap acceptance and rejection behavior, and yielding behavior towards traffic in circle and also towards pedestrians at roundabout crossings. Results showed that roundabout geometry significantly affects speed selection behavior of drivers as they approach the yield line. In terms of gap behavior, drivers tended to accept smaller gaps at multilane roundabouts, three-legged roundabouts, roundabouts in rural areas, and roundabouts that were not on interchange compared to their counterparts. Yield rates towards both pedestrians and traffic in circle tended to be the lowest on roundabouts located on interchange. In terms of safety, roundabouts were found to reduce crash severity and reduce the proportion of certain crash types. Safety analysis was carried out at various levels of detail including naïve before-after, empirical Bayes (EB) method, and cross-sectional analysis. However, all the approaches showed that conversion of an intersection to a roundabout significantly increased the number of total crashes but reduced the number of fatal and injury crashes. These trends were generally comparable to prior roundabout study carried out by Michigan Department of Transportation (MDOT) published in 2011. Safety performance functions (SPF) were also developed for roundabouts based on the number of approach legs and number of circulating lanes. Roundabouts were also found to have significant operational and environmental benefits in terms of reduced delay, better level of service (LOS), and fuel savings. Lastly, economic analyses were also conducted to develop benefit-cost curves and estimate benefit/cost ratios for select roundabouts. The research results are intended to guide MDOT and other road agencies in deciding the need to convert an existing intersection to a roundabout.

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Evaluating the Performance and Safety Effectiveness of Roundabouts- An Update

FINAL REPORT

January 30, 2023

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EXECUTIVE SUMMARY

Since the installation of its first modern roundabout in 1996, the state of Michigan has seen a considerable increase and there are now more than 180 roundabouts, with additional roundabouts currently in the planning, design, and construction phase by various roadway agencies. The Michigan Department of Transportation (MDOT) conducted an initial study on the safety effectiveness of roundabouts that was published in 2011. Since the completion of that study, numerous new roundabouts have been constructed throughout the state. Consequently, this study was aimed at updating this evaluation based upon these more recent installations. A series of analyses were carried out in three phases. First, field data were collected to investigate driver behavior under various contextual conditions. Secondly, traffic, crash, and geometry data were collected for roundabouts throughout the state for the purposes of a comprehensive safety analysis. Finally, a series of evaluations were conducted to compare the operational and environmental benefits of converting conventional intersections to roundabouts. The study culminates in the development of guidance for MDOT and other road agencies to inform planning and design decisions for future roundabouts.

Detailed field data were collected at 18 roundabout sites across Michigan. These field studies included the collection of data regarding: (1) speed profiles for vehicles as they entered the roundabouts; (2) gap acceptance and rejection behavior by these drivers; and (3) yielding behavior towards other vehicles in the traffic circle as well as towards pedestrians intending to cross the roundabout. Speed selection was found to vary based upon traffic volumes, type of vehicle, and posted speed limit on the approach. In general, reduction in speeds were more pronounced within 200 ft of roundabout yield line, and drivers generally traveled at higher speeds unless they were required to yield by pedestrians, bicyclists, or cross-traffic.

In terms of gap acceptance behavior, the mean accepted gap ranged from 3.3 s to 6.9 s for passenger cars, while the mean rejected gap for passenger cars ranged from 1.4 s to 3.5 s. The critical gap, which is defined as the minimum gap in the traffic circle that drivers are willing to accept, varied based upon the number of circulating lanes, number of approach legs, and contextual characteristics. Multilane roundabouts had lower critical gaps, while roundabouts with three legs showed lower critical gaps than four-legged roundabouts. Roundabouts in urban areas, or roundabouts located on ramp terminals also had higher critical gaps. Yielding behavior of drivers towards traffic in the circle was also investigated which showed yield rates exceeding 80% across all sites. Roundabouts located on interchanges showed lower yield rates compared to other roundabouts. The likelihood of drivers yielding to traffic in circle was found to be affected by traffic volume, roundabout diameter, context of roadway, type of roundabout, presence of roundabout warning sign and right bypass lane, and the time to traffic in circle.

Yielding rates towards pedestrians at crossings were found to be significantly lower on roundabouts located at interchanges, where less than 45 percent of drivers yielded. Sites with pedestrian hybrid beacons (PHB) exhibited yield rates exceeding 90 percent, although these trends were comparable to other sites without PHB. Installing a PHB on pedestrian crossings on exit ramps may help in increasing yielding rates towards pedestrians since drivers do not usually expect pedestrian presence at ramp terminals.

Comprehensive safety analyses of Michigan roundabouts were also conducted at various levels of detail. Companion control intersections were identified for each of the roundabouts as a reference group. Traffic volume, crash, and roadway geometry data were collected from various sources and integrated to create several datasets to conduct a before-after analysis. The aggregated data results showed that crash severity was significantly reduced after roundabout construction, as were the frequencies of several severe crash types, such as angle collisions. Naïve before-after analyses showed total crashes increased after roundabout conversion while injury crashes were significantly reduced.

Subsequently, an empirical Bayes (EB) analysis was conducted to estimate crash modification factors (CMF) for these roundabout conversions. Across all roundabout types, the results showed that total crashes increased while fatal and injury (FI) crashes decreased after roundabout construction. The resultant CMFs were 2.10 and 0.92 for total crashes and FI crashes, respectively. For single-, double-, and triple-lane roundabouts, the corresponding CMFs were 1.03, 2.13, and 3.07 for total crashes, respectively. For FI crashes, the analogous CMFs were equal to 0.60, 0.94, and 1.26, respectively. Converting a signalized intersection to a single- or double-lane roundabout had CMFs of 1.92 and 0.81 for total crashes and FI crashes, respectively. Similarly, converting stop-controlled intersection to single- or double-lane roundabouts had CMFs of 1.29 and 0.76 for total and FI crashes, respectively. Triple-lane roundabouts showed significantly higher increases in crashes, though the sample of such locations was significantly smaller than the other roundabout types.

Cross-sectional analyses were also conducted which showed roundabouts, on average, tended to have 58% (CMF = 1.58) more crashes than conventional intersections. Fatal and injury crashes on roundabouts were 27% lower (CMF = 0.73) than conventional intersections. Lastly, safety performance functions (SPF) for roundabouts were also developed separately for three-legged roundabouts, four-legged roundabouts with one circulating lanes, and four-legged roundabouts with two circulating lanes.

From an environmental and efficiency standpoint, roundabouts have also generally been shown to reduce user delay and reduce fuel consumption, thereby reducing emissions. To that end, 15 roundabout-intersection pairs were identified and the differences in delay and level of service were investigated. Results showed that roundabouts were effective in reducing delay by 57-67 percent. Additionally, it was found that the conversion of conventional intersection to a roundabout can result in nearly $2.3 per vehicle per year savings in terms of fuel consumption, and $67 per vehicle per year in terms of delay savings. Economic analyses showed the benefits from roundabout conversion significantly outweigh the construction costs. Benefit-cost curves were also developed as a function of traffic volume. These provide a resource for MDOT and local agency staff to assess the suitability of roundabouts at candidate locations.

Overall, the results showed that roundabouts have been effective in improving safety, reducing delay, and providing environmental benefits as compared to conventional stop-controlled or signalized intersections. It should be noted that roundabout construction has generally been associated with an increase in the total number of crashes, although the number of injury crashes and severity of crashes are reduced, generally resulting in a net benefit. Converting an intersection to a roundabout is expected to be most beneficial at intersections that experience a disproportionately high number of severe crashes (e.g., angle and head-on/left-turn collisions). Roundabouts also generally reduce intersection approach and entry speeds, contributing to the reduction in severe crashes. Lastly, roundabouts located at interchanges may need special considerations since these tended to have higher approach speeds and the lowest yield rates towards both pedestrians and other vehicles.

1 INTRODUCTION

Roundabouts are a specific type of circular roadway junction, where traffic yields prior to entering the intersection, then moves in a counter-clockwise direction around the circle until departing in the desired direction of travel. Roundabouts are becoming increasingly popular worldwide including the United States. The number of roundabouts in the US has increased from fewer than 50 in 1997 (Jacquemart, 1998) to more than 8,800 as of 2021 (Kittelson & Associates, Inc., n.d.). The state of Michigan (MI) mirrors these trends. The first modern roundabout in MI was installed in 1996 Road Commission for Oakland County at the intersection of Tienken, Washington, and Runyon Roads in Rochester Hills. Since that time, roundabout construction has increased substantially and a preliminary search identified nearly 180 roundabouts at various locations throughout the state. Figure 1 shows the location of all 180 known roundabouts in the state of Michigan. Beyond these sites, there are a number of additional roundabouts in the planning, design, or construction phase by Michigan Department of Transportation (MDOT) and other road agencies.

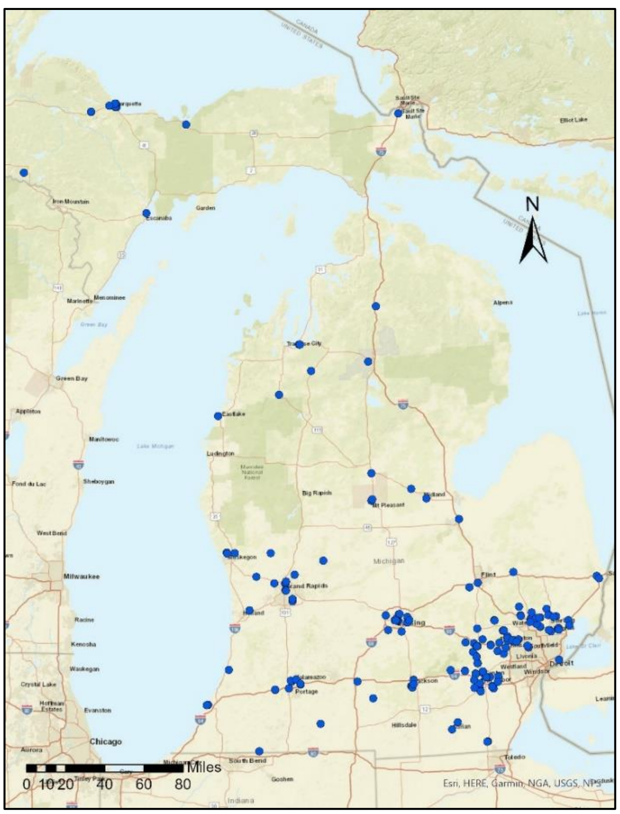

Figure 1 Roundabout Locations in Michigan

These trends have generally been driven by research that has shown roundabouts to improve both safety and operational performance under various contexts, as well as long-term cost savings, reduced fuel consumption and emissions due to less time spent stopping or idling as compared to a typical signalized or stop-controlled intersection (L. A. Rodegerdts et al., 2010). While a four-leg intersection has 32 points of conflict, a roundabout only has eight. Similarly, pedestrian conflicts are reduced from 24 at a four-legged intersection to only 8 at a roundabout. Figure 2 compares the number of conflict points for both vehicles and pedestrians on a traditional four-legged intersection and a roundabout (Ihnen, 2013). It is worth noting that all the vehicle conflict points in a roundabout are either diverging or merging as opposed to crossing type conflict points that generally occur in traditional signalized or stop-controlled intersections. These advantages are due to geometric characteristics that essentially eliminate several high-risk conflict types (e.g., head-on and angle collisions) and require drivers to reduce their speeds.

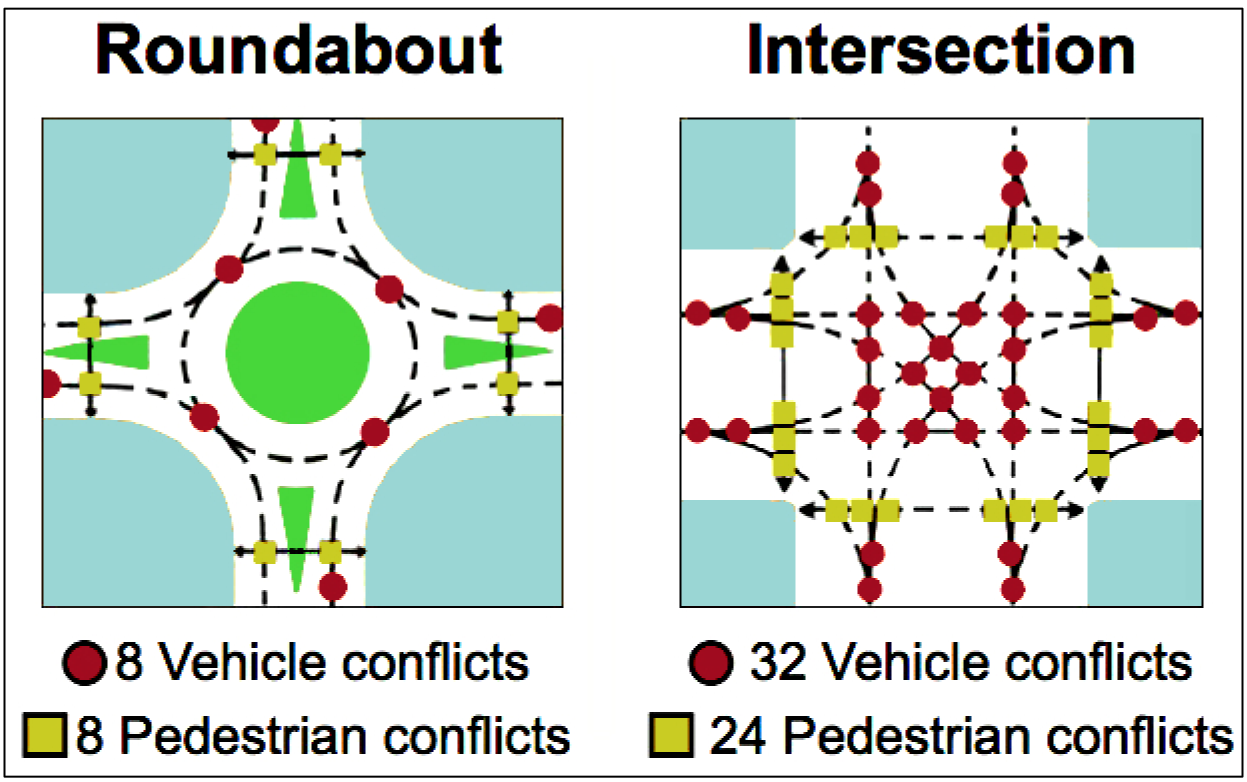

Figure 2 Conflict Points in a Roundabout and Traditional Intersection *(Ihnen, 2013)*

1.1 Problem Statement and Study Objectives

The state of Michigan has conducted two separate studies on roundabouts. The first study published in 2011 showed an average reduction of 41.7% in injury crashes at sites that were converted to roundabouts, though total crashes actually increased by 34.6% (Bagdade et al., 2011). It is important to note that this increase was largely driven by roundabouts with two and, particularly, three circulating lanes. A parallel MDOT study examined driver behavior, familiarity, and understanding of roundabouts, highlighting issues of concern for future design and construction (Savolainen et al., 2011). This study also involved a comprehensive evaluation of UD-10 crash reports, as well as companion field studies of driver behavior, providing important complementary information to guide subsequent investment decisions.

However, since the completion of these two MDOT research projects, a significant number of roundabouts have been constructed throughout Michigan. A number of additional studies have also been conducted, including several NCHRP projects (Ferguson et al., 2019; Savolainen et al., 2011; Schroeder et al., 2017), as well as state-level studies in Arizona (Mamlouk & Souliman, 2018), Georgia (Gbologah et al., 2019), Minnesota (Leuer, 2017), Missouri (Claros et al., 2018), Pennsylvania (Coffey et al., 2016), and Wisconsin (Bill et al., 2011). Further, there have been important advancements in best practices for both geometric design and the provision of traffic control measures, such as signage and pavement markings. This includes the introduction of pedestrian hybrid beacons (PHBs) and rectangular rapid-flashing beacons (RRFBs) in order to accommodate the needs of non-motorized users, particularly those who are visually impaired. In addition, performance measures at select locations have either improved or declined for various reasons.

As such, further research is warranted to evaluate both the short-term and long-term performance of Michigan roundabouts, culminating in guidance for future investment decisions in roundabouts by both MDOT and other county and local road agencies throughout the state. To that end, the primary goal of this research is to provide MDOT with updated data-driven support tools and other guidance to inform the installation of future roundabouts. The specific objectives of this research are as follows:

- Determine national/international existing understanding of roundabout efficacy relative to environment, operations, safety, maintenance and cost.
- Determine the environmental efficacy of Michigan roundabouts.
- Determine the operational costs, efficacy and performance of Michigan roundabouts.
- Determine safety efficacy/performance of Michigan roundabouts including vehicles and pedestrians.
- Determine the typical maintenance costs, best practices and make recommendations.
- Observe/analyze roundabout operations, including speeds, truck movements and pedestrian activity.
- Identify characteristics that influence benefit/cost for roundabouts.
- Compare results to prior study results where applicable (H-U0316.02).

1.2 Task Summary

In order to achieve the above stated research objectives, the following tasks were performed. Detailed description of these tasks has been provided in the subsequent chapters of this report.

- <u>Literature Review:</u> A comprehensive state-of-the-art literature review was carried out to investigate the safety, operational and environmental efficacy of roundabouts.

- <u>Operational Performance of Roundabouts</u>: This chapter details the methodology adopted to select sites for field data collection, data collection procedure, followed by data extraction and analyses results. Analysis was carried out to investigate driver's speed selection behavior, gap acceptance behavior, and yielding behavior.

- <u>Safety Performance of Roundabouts</u>: The safety performance of roundabouts is investigated by various analysis methods including simple before-after comparison of crash frequencies, empirical Bayes or EB method, and cross-sectional method. Safety Performance Functions (SPF) for roundabouts are also developed.

- <u>Level of Service and Economic Analysis of Roundabouts</u>: The effect of roundabout construction in reducing delay and improving the level of service (LOS) were investigated. The environmental benefits in terms of fuel savings and reductions in emissions were also analyzed.

- Conclusions and Recommendations: This chapter presents the conclusions of the research followed by recommendations for the transportation agencies.

2 LITERATURE REVIEW

Since 1996, the number of roundabouts in Michigan have increased considerably to at least 180, and a number of additional roundabouts in the planning, design, or construction phase by MDOT and other road agencies. Consequently, there is ample opportunity to build upon the insights from prior research that has been conducted both in Michigan and nationwide. This includes investigating several emerging areas where gaps have been identified in the research literature. To this end, the following sections provide an extensive summary of previous literature related to safety, operational, environmental, and economic benefits of roundabouts.

2.1 Safety Performance of Roundabouts

2.1.1 Motorists

At a national level, several important guidance documents have been developed to support the strategic implementation of roundabouts across the United States. This began with *NCHRP Synthesis 264: Modern Roundabout Practice in the United States* (Jacquemart, 1998) which provided general guidance to assist road agencies in the design, construction, operation, and maintenance of roundabouts. At the time of its publication, research experience in the US was somewhat limited as only 38 modern roundabouts had been constructed as of October 1997, though early research had shown crash reductions in the range of 60 to 70% (Flannery & Datta, 1996). Consequently, this guidance document was largely based on international experience, which showed significant crash reductions for total and injury crashes in Australia (Austroads, 1993), Norway (Giaever, 1992), the Netherlands (Schoon & Van Minnen, 1994) France (Guichet, 1997), and England (Lalani, 1975).

Subsequently, the first edition of *Roundabouts: An Informational Guide* was published in 2000 as part of a project funded by the Federal Highway Administration (FHWA) (Robinson et al., 2000). This guidance document was also largely based on international research, but included some early work that had been conducted at some locations in the US. This document served as the basis for MDOT’s *Roundabout Guidance Document*, which was published in November 2007 (Bott et al., 2007).

Subsequent evaluations were conducted in several states across the US. A Nevada study showed low volume (under 10,000 ADT) sites to experience 59% fewer total crashes and 83% fewer injury crashes (Nambisan & Parimi, 2007). Moderate (10,000 to 20,000 ADT) volume sites experienced 22% fewer total crashes and 42% fewer injury crashes while high-volume (over 20,000 ADT) roundabouts experienced 24.2 percent more crashes, but 17 percent fewer injury crashes. A 2006 study used both the EB method and a cross-sectional analysis to assess the safety performance of roundabouts that were converted from signalized intersections. The EB evaluation examined data from 28 roundabout conversions from throughout the United States while the cross-sectional analysis compared data between 42 signalized intersections and 26 newly constructed roundabouts. The results of both analyses showed fewer total and injury crashes at roundabouts as compared to signalized intersections, with the reduction in injury crashes being significantly greater in magnitude (Gross et al., 2013).

The first large-scale evaluation of modern roundabouts in the US was conducted as a part of NCHRP Project 3-65 and is documented in NCHRP Report 572. This project involved an intersection-level evaluation of 90 roundabouts and an approach level analysis of 139 intersection legs. The results showed roundabouts to effectively improve intersection performance across a variety of performance measures (L. A. Rodegerdts et al., 2007). This included significant reductions in traffic crashes and injuries, motorist delay, and the associated costs borne by both road users and transportation agencies. These benefits were most pronounced at intersections that were converted from signalized or two-way stop-control. Roundabouts were also shown to exhibit a 40% to 50% reduction in pedestrian crashes compared to conventional intersections, though bicycle-involved crashes generally increased by a factor of 1.8 to 4.5 at roundabouts.

The results from this project were included as a part of NCHRP Report 672, the second edition of *Roundabouts: An Informational Guide*, which was jointly funded by the FHWA as a part of NCHRP Project 3-65A (L. A. Rodegerdts et al., 2010). In addition to detailing the safety benefits, this document provided guidance related to common concerns associated with the design, operation, and maintenance of roundabouts. Much of this information was subsequently included in MDOT's *Roundabout Design Aid* (MDOT, 2019).

In addition to the design guideline, MDOT also evaluated the performance of the roundabouts in Michigan. As described previously, MDOT utilized the EB method to develop a series of SPFs for roundabout conversions and the roundabout itself. In this report, the SPFs were first developed based on the major and minor road annual average daily traffic volume (AADT) to determine the changes in crashes after conventional intersections were converted to roundabouts. The CMFs were subsequently calculated based on the SPFs and calibration factors. The findings indicated that the total crashes increased, especially in the sites where the signalized intersections converted to three-lane roundabouts. However, a reduction in injury crashes was observed regardless of the site types. The reduction was more pronounced for the one- and two- lanes roundabouts converted from signalized intersections (Bagdade et al., 2011).

As part of this work, MDOT also developed roundabout-only SPFs. The total entering AADT, number of circulating lanes, number of approach legs, environment (i.e., urban/rural), and whether or not the intersection was an interchange were the variables considered in developing SPFs. The final equation of the SPF is shown in Equation 1 and was applied to both total crashes and injury crashes. The estimated parameters indicated that the single-lane roundabouts experienced fewer crashes than the double-lane roundabouts, and the roundabouts at an interchange tended to have more crashes than other roundabouts (Bagdade et al., 2011).

$$N = exp^{\beta_0} * (AADT)^{\beta_1} * exp^{\beta_2 * type + \beta_3 * IC} \quad \text{Equation 1}$$

Similarly, the Oregon Department of Transportation (ODOT) developed roundabout-only SPFs as well based on 21 four-leg single-lane roundabouts and five years of crash data. Unlike MDOT, only entering AADT was included in the SPFs. The general equation of SPF developed by ODOT is displayed in Equation 2 (Dixon & Zheng, 2013).

$$N = \frac{exp^{\beta_0 + \beta_1 * (AADT)^2}}{5} \quad \text{Equation 2}$$

Where

N = predicted average crash frequency, crashes/years;

AADT = Total entering AADT; and

β_0, β_1 = Estimated parameters.

Except for the roundabout-only model, the roundabouts were compared to stop controlled and signalized intersections by a cross-sectional method with the SPFs for stop controlled and signalized intersections were obtained from the Highway Safety Manual (HSM). The results showed that the roundabouts experienced substantially fewer crashes than the traditional intersections with the same entering AADT (Dixon & Zheng, 2013).

Based upon the positive results experienced in the US, the FHWA identified roundabouts as one of the Office of Safety's Proven Safety Countermeasures that were included in the Every Day Counts 2 campaign for Intersection & Interchange Geometrics. Consequently, to address specific questions on how to tailor certain aspects of their design to better meet the needs of a growing number and diversity of stakeholders, the FHWA funded *Accelerating Roundabout Implementation in the United States*. This project resulted in a seven-volume collection that addressed several pressing issues of National significance, including enhancing safety, improving operational efficiency, considering environmental effects, accommodating freight movement and providing pedestrian accessibility (Findley et al., 2015a, 2015b; L. Rodegerdts et al., 2015; L. A. Rodegerdts et al., 2015; Salamati et al., 2015; Schroeder et al., 2015; Steyn et al., 2015).

In 2017, NCHRP Report 834 (Crossing Solutions at Roundabouts and Channelized Turn Lanes for Pedestrians with Vision Disabilities: A Guidebook) was published, which detailed issues that visually impaired pedestrians face when accessing and crossing intersections, particularly roundabouts (Schroeder et al., 2017). This included a guidance document that provided information to establish safety crossings for such individuals and notes the two predominant factors contributing to issues at such locations include (1) inconsistencies between lane use markings on the approach and those within the circulatory roadway, and (2) insufficient channelization for drivers when being shifted from the inside lane to the outside lane to exit. The guidebook details alternatives such as the PHBs and RRFBs that have been installed in Oakland County, as well as additional strategies that are less cost-prohibitive and may be of interest to MDOT and other Michigan road agencies. This includes a methodology for evaluating treatment alternatives, as well as wayfinding accommodations, in addition to guidance on the feasible range conditions under which these treatments have been effective (Schroeder et al., 2017).

Meanwhile, roundabouts were widely examined across various states. A before-and-after study done by MnDOT in 2017 found that 144 intersections in Minnesota constructed since 1995 had an overall 15.6% increase in total crashes, an 87% reduction in the fatal crash rate, and 84% reduction of severe crashes after roundabout installation. It was found that these roundabouts also had a 36% reduction in right angle crashes and a 71% reduction in head-on left turn crashes. In single lane roundabouts, there was an 89% reduction in fatal crashes and an 83% reduction in serious injury crashes, causing an 86% reduction in severe crashes. There was also a 68% reduction in right angle crashes, an 83% reduction in head-on left turn crashes, and a 51% reduction in multi-vehicle crashes. In double lane roundabouts, there was a 100% reduction in serious injury crashes and severe crashes. While there was a 25% reduction in head-on left turn crashes, there was a 133% increase in right angle crashes, a 2,979% increase in sideswipe same direction crashes, and a 148% increase in multi-vehicle crashes. This caused an increase of 146% in the total crash rate. In unbalanced lane roundabouts, there was a 78% reduction in serious injury crashes and a 78% reduction in severe crashes. There was also a 25% reduction in right angle crashes and an 83% reduction in head-on left turn crashes, however, there was a 774% increase in sideswipe same direction crashes and a 22% increase of multi-vehicle crashes (Leuer, 2017).

Similarly, another study on 17 roundabouts across five cities in Arizona comparing average crash rates single lane roundabouts found that these roundabouts saw a decrease of percent per year and a decrease of 19 percent per million vehicles in average crash rate after installation. In the same study, it was also found that double lane roundabouts saw an increase of 62% per year and an increase of 55 percent per million vehicles in average crash rate after installation. Out of the 17 roundabouts analyzed in this study, 11 were single lane roundabouts and 6 were double lane roundabouts (Mamlouk & Souliman, 2018). A report from Louisiana also utilized EB analysis to evaluate the safety performance of 18 single-lane roundabouts. The reduction in total crashes was more pronounced at the roundabouts that converted from the stop sign on minor road intersection (49% crash reduction). The findings also stated that the overall injury crashes reduced after conversations by lower driving speed and eliminating certain types of collisions (i.e., left turn, head-on, right angle and sideswipe collisions) (Sun & Rahman, 2019).

Additionally, a study from the state of Georgia analyzed a total of 23 three- and four-leg roundabouts that had been converted from stop controlled and signalized intersections by extended EB method. For this study, the annual traffic volume was primarily utilized to estimate the safety performance functions (SPFs) for each analysis year. Ultimately, the final parameters of SPFs were estimated as a function of time. Therefore, unlike the typical EB method, this study considered the time-dependence for SPFs. Additionally, the corresponding crash modification factor (CMFs) were developed for all roundabouts, three- and four-leg intersections individually in terms of crash types. It found a reduction of 37 to 48 percent in total number of crashes and a reduction of 51 to 60 percent in injury and fatal crashes at four-leg roundabouts converted from the stop-controlled intersections. Additionally, the overall safety performance of roundabouts improved as well. The total number of crashes were reduced by 56 percent and the injury/fatal crashes were reduced by 69 percent regardless of the number of legs (Gbologah et al., 2019).

As of October 2020, the most recent national evaluation of roundabouts is presented in NCHRP Report 888, which details the work conducted as a part of NCHRP Project 17-70. This project involved the development of updated predictive safety tools for roundabouts using data from 355 roundabouts across 11 states. A series of SPFs and CMFs were developed through a cross-sectional analysis that considered both unique characteristics and common features of each roundabout. The CMFs provide estimates as to the impacts of various roundabout design elements (e.g., inscribed diameter, entry width) on specific types and severities of crashes. These models were developed at three levels of detail, allowing for estimation of safety performance at the (a) planning-level, (b) intersection-level, and (c) individual approach leg-level. Considering the purposes of this report, the following sections will discuss the safety performance at the planning-level and intersection-level in detail (Ferguson et al., 2019).

For (a) planning-level, separate SPFs were developed for rural and urban, as well as single- and multilane roundabouts across different injury severity levels (i.e., total crashes, fatal and injury crashes, and PDO crashes). The following equations show the general SPFs for different categories. The magnitudes of estimated parameters varied for each severity level while the signs remained the same. Based on these parameters, the major/minor AADT, four-leg, or multilane roundabouts had positive impacts on the roundabout crashes (Ferguson et al., 2019).

1. Rural roundabouts:

$$N = exp^{\beta_0 + STATE} * MAJADDT^{\beta_1} * MINAADT^{\beta_2} * exp^{\beta_3 * NUMBERLEGS + \beta_4 * CIRCLANES} \qquad \text{Equation 3}$$

2. Urban single-lane roundabouts:

$$N = exp^{\beta_0} * MAJADDT^{\beta_1} * MINAADT^{\beta_2} * exp^{\beta_3 * NUMBERLEGS} \qquad \text{Equation 4}$$

3. Urban multilane roundabouts:

$$N = exp^{\beta_0} * MAJADDT^{\beta_1} * MINAADT^{\beta_2} * exp^{\beta_3 * NUMBERLEGS} \quad \text{Equation 5}$$

Where

N = predicted average crash frequency, crashes/yrs;

STATE = an additive intercept term dependent on state;

MAJAADT = total entering AADT on major road;

MINAADT = total entering AADT on minor road;

NUMBERLEGS = 1 if a 3-leg roundabout; 0 if 4 legs;

CIRCLANES = 1 if a single-lane roundabout; 0 if more than 1 circulating lane; and

$\beta_0, \beta_1, \beta_2, \beta_3, \beta_4$ = Estimated parameters.

For (b) intersection-level, eight base SPFs were developed for three- and four-leg roundabouts across different injury severity levels (i.e., fatal and injury crashes and PDO crashes) and the number of circulating lanes (i.e., one and two circulating lanes). For each base SPF, only entering AADT and whether a roundabout was in a rural area were included. In addition, multiple CMFs were developed to calibrate the SPFs predicted results when the non-base conditions are interested. Ultimately, the predicted average crash frequency was be estimated following Equation 6 and 7 (AASHTO, 2010; Ferguson et al., 2019).

$$N_m = C * N_{SPF,m} * (CMF_1 * CMF_2 * \ldots * CMF_n \quad \text{Equation 6}$$

$$N_{SPF,M} = exp^{\beta_0 + \beta_1 * LN\left(\frac{ENTAADT}{1000}\right) + \beta_2 * RURAL} \quad \text{Equation 7}$$

Where

N_m = predicted average crash frequency for roundabout with m legs (m = 3, 4), crashes/yr;

C = calibration factor for specific jurisdictions represented in the database;

$CMF_1 * CMF_2 * \ldots * CMF_n$ = CMF for traffic characteristic, geometric element, or traffic control n.

$N_{SPF,m}$ = predicted average crash frequency for base condition on all legs for roundabout with m legs (m = 3,4), crashes/yr

ENTAADT = total entering AADT;

RURAL = 1 if area is rural, 0 otherwise; and

$\beta_0, \beta_1, \beta_2, \beta_3, \beta_4$ = Estimated parameters.

Different CMFs were developed for each condition. For example, a CMF for inscribed circle diameter (CMF_{ICD}), a CMF for outbound-only leg (i.e., if a leg serves as a ramp terminal, CMF_{outbd}), a CMF for right-turn bypass (CMF_{bypass}), and a CMF for the presence of driveways or unsignalized access points on a leg within 250 ft. of the yield line (CMF_{ap}) were included in the fatal and injury (FI) crash frequency prediction model, one circulating lane. Only CMF_{ap} was included in the PDO crash frequency prediction model, one circulating lane. For two circulating lane models, CMF_{outbd}, CMF_{bypass}, CMF for entry width (CMF_{ew}), and a CMF for circulating lane (CMF_{cl}) were applied in the FI crash model, while the CMF_{ew} and CMF_{cl} were applied to the PDO crash model.

Additionally, the report also examined the trends of each CMF. It found that the CMF_{ICD} and CMF_{ap} increased with increasing inscribed circle diameter and number of access points while CMF_{ew} decreased as the entry width increased. The CMF_{outbd} suggested that a roundabout with a leg serving as a ramp terminal had fewer crashes than in a roundabout at which all legs serve two-way traffic flow. CMF_{bypass} suggested that the presence of a right-turn bypass reduced the crash frequency. Finally, CMF_{cl} suggested that a leg with one circulating lane experienced fewer crashes than if there were two circulating lanes.

Lastly, NCHRP Project 03-130 has been recently completed and involves the development of the third edition of *Roundabouts: An Informational Guide*, however, the final report has not published yet (Kittelson & Associates, Inc., 2022). It is expected that this report will include details of specific guidance that has been updated from the second edition, as well as the documentation of any inconsistencies with relevant documents, such as *A Policy on Geometric Design of Highways and Streets* (AASHTO, 2018) and the *Manual on Uniform Traffic Control Devices* (Federal Highway Administration, 2009).

2.1.2 Non-Motorized (Vulnerable) Users

In addition to safety performance for motorists, the safety performance for vulnerable road users is also essential for the roundabout research. However, the safety of pedestrians and bicyclists at roundabout crossings is under-researched. Pedestrian and bicycle activity levels are relatively low in the United States which results in fewer crashes that involve pedestrians or bikes. Thus, much of the research is generated from Europe and Australia. This makes results transferable to the US complicated due to differences in geographical contexts, traffic laws, and driver behavior. Nevertheless, studies have shown pedestrian and bicyclist safety to improve at roundabouts due to a reduction in conflict points compared to a typical four-legged intersection (Daniels et al., 2008; Maycock & Hall, 1984). For example, in the United Kingdom, the rate of pedestrian involved and bicyclist involved crashes at a conventional roundabout is 0.45 and 2.91 crashes per million trips, respectively, compared to 0.67 and 1.75 crashes per million trips at signalized intersections, respectively (Maycock & Hall, 1984).

The extant literature has generally shown that the roundabouts provide a safer environment for non-motorized road users, however, the conversion of a signalized intersection to a roundabout may actually increase frequency of crashes involving pedestrians or bicyclists. Studies have also shown that multilane roundabouts may not be the best intersection design compared to the single-lane roundabouts in cases where multimodal activity is prevalent as the complexity of navigation through roundabouts increases with the presence of multiple lanes (Arnold et al., 2013; Aumann et al., 2017; Patterson, 2010; L. A. Rodegerdts et al., 2010; Wilke et al., 2014).

2.2 Operational Performance of Roundabouts

2.2.1 Motorists

In addition to improving safety, roundabouts also have positive operational impacts compared to conventional intersections. To estimate capacity, delay and level-of-service at roundabouts, three methods were typically applied: regression analysis, gap acceptance analysis, and simulations (e.g., RODEL or SIDRA) (Flannery et al., 1998; Qu et al., 2014; L. A. Rodegerdts et al., 2007, 2010; Sisiopiku & Oh, 2001). In 1999, a study was conducted to compare the operational performance of four-leg roundabouts with various "traditional" intersections, such as yield control, two- and four-way stop controlled, and signalized intersections by using SIDRA. After comparing the performance in terms of delays and capacity, the one-lane roundabouts showed similar performance with signalized intersections, but the yield and two-way stop-controlled intersections were found to be better alternatives compared to roundabouts under light traffic volumes. Two-lane roundabouts were shown to perform better than other types of intersections with the same number of lanes and also outperformed with heavy left-turn demand in terms of capacity and delay.

The signalized intersections performed better over roundabouts with three-lane approaches. However, the increased capacity was observed with two- or three-lane roundabouts compared to a signalized intersection, regardless of the total entering flow (Sisiopiku & Oh, 2001).

Other studies also indicated that roundabouts have better performance than other types of intersections (IIHS, 2022; Retting et al., 2002, 2006). One study evaluated three roundabouts converted from stop-sign controlled intersections in Kansas, Maryland and Nevada finding that vehicle delay decreased by 13 to 23 percent and vehicle stops were reduced by 14 to 37 percent (Retting et al., 2002). Following this, another similar study evaluated three roundabouts converted from stop-sign controlled and signalized intersections in New Hampshire, New York and Washington state. In this study, vehicle delay during peak hours was reduced by 83 to 93 percent and volume-to-capacity ratio (i.e., measurement of traffic congestion) was reduced by 54 to 84 percent (Retting et al., 2006). Furthermore, an extensive review of 11 roundabouts in Kansas also proved that the overall operational performance improved significantly after converting from stop-controlled and signalized intersections in terms of delay, queue length and proportion of vehicle stops (Russell et al., 2005).

Additionally, roundabouts have also been shown to reduce delays by about 75 percent in comparison to previous "traditional" intersection configurations. *NCHRP Report 572* showed that the number of lanes produced the greatest impacts on roundabout capacity and delay while geometric adjustments, such as changing the lane width or entry/exit radius, did not provide significant differences (L. A. Rodegerdts et al., 2007). The 2011 Michigan evaluation study also examined traffic operations at five roundabouts, which included a mix of single-lane, multi-lane, and interchange roundabouts. Before-conversion delay was estimated using Synchro and Highway Capacity Software (HCS) while post-conversion roundabout performance was evaluated using RODEL. Delay was found to be consistently reduced across all sites, with the percent reduction ranging from 74% to 94% (Bagdade et al., 2011).

Additionally, a before and after comparison was also conducted on two roundabouts in Bellingham, Washington, with the study sites undergoing conversion from two-lane stop-controlled intersections. The study discovered that the proportion of vehicle stops decreased by 35 and 45 percent for the two sites and delay on the minor roads decreased by 33 and 90 percent. However, unlike other studies, the overall delay showed an increasing trend (12 and 22 percent) for these two roundabouts. The study stated that the increment was caused by the substantial increase in peak hour left-turning traffic on the major road (Hu et al., 2014).

Moreover, a recent study compared the vehicle capacity, average speed, number of stops, and delay time between roundabouts and conventional signalized intersections under high and low volume scenarios. The researchers developed six junction models for each scenario. The results demonstrated that the roundabouts had higher vehicle capacity than signalized intersections, especially under high volume conditions. The improvements in average speed and delay time were also found to be more pronounced in roundabouts under heavy traffic. However, in the same situation, roundabouts had a higher possibility of generating more vehicle stops than signalized intersections (Zhou et al., 2022).

There are several other operational issues that research has suggested warrant further investigation, including an interaction between approaching flow and general driver behavior when approaching and navigating roundabouts. For example, overestimating capacity and underestimating delay and queue length would be observed if neglecting the approach flow interactions, especially for multilane with a balanced flow (1997). A case study from Alaska showed increases in delay and queue were observed if one entrance at a multilane roundabout experienced higher traffic flow than others (Akcelik et al., 1997; Chen & Lee, 2016). In addition, drivers in the US appear to use roundabouts less efficiently than models suggest is the case in other countries around the world (MDOT, 2020). One evaluation showed that American drivers reacted differently than Australian drivers under similar conditions and tended to accept smaller gaps in the traffic stream on entry at roundabouts as compared to stop-controlled intersections (Flannery & Datta, 1996). This may be partially explained by inexperience with navigating roundabouts and it may be reasonably expected that this may change over time. This is an issue that can be investigated through additional field studies of driver behavior.

Another area that has emerged as a concern is driver speed selection during the approach/entry, traversal, and exit from roundabouts. For example, NCHRP Report 888 explored the relationship between speed and crashes at roundabouts. Fastest path radii were calculated for each approach at 32 roundabouts for right-turning, left-turning, and through-traveling paths in order to predict speeds. This preliminary analysis indicated more research is needed to understand the potential relationship, though the results did show a relationship between posted speed limits and single-vehicle crashes. It was suggested that future research should focus on further examining the relationship using both measured and theoretical speeds (Ferguson et al., 2019).

2.2.2 Non-Motorists

Bicycle and pedestrian crossing behavior at roundabouts is also crucial for the roundabout evaluation. A recent study reviewed 49 previous research studies across various countries finding that that higher speed, multilane roundabouts, and on-roadway bike lanes through roundabouts tended to have the worst bicycle safety performance. The entering drivers were more likely to fail to yield to the cyclists if the cyclists operated as vehicles at roundabouts. The research suggested that a separate cycle path around the roundabout should be provided to lower the risk of involving in crashes (Poudel & Singleton, 2021). Pedestrian crossing behavior at roundabouts in the United States remains unclear. The limited research available indicates that roundabouts improve pedestrian safety and that pedestrian delay had a positive relationship (increased) with traffic volume (Rouphail et al., 2005; Stone et al., 2002).

In the absence of sufficient crash data, alternate data are being collected and analyzed to evaluate pedestrian and bicyclist safety at roundabouts, including surveys and the analysis of video data such as vehicle speeds, trajectory, and their interaction with non-motorized road users. A study conducted at 14 approaches across 7 roundabouts in the US found no substantial safety issues for non-motorists at roundabouts in terms of collision or conflicts (Harkey & Carter, 2006). However, the study found exit legs to be of greater risk to pedestrians than entry legs due to lower yielding rates at exits. Additionally, one -lane approaches were safer for pedestrians compared to two-lane approaches. At roundabouts with a single lane approach, 17% of the motorists did not yield for a crossing or waiting pedestrian while at two-lane approaches, 43% of the motorists did not yield. The study also found a few events of wrong-way driving by bicyclists particularly when entering the roundabout from the exit leg. The study recommended that the design of exit legs should be improved to ensure adequate sight lines and minimum vehicle speeds to improve safety of non-motorized users.

2.3 Environmental and Economic Performance of Roundabouts

Beyond safety and operations, recent research has provided guidance as to simple methods for estimating environmental impacts of roundabouts. An FHWA study resulted in vehicle activity and emissions models for roundabouts and signalized intersections based on Portable Emissions Measurement System (PEMS) data. These models are implemented in a spreadsheet-based emissions computational engine, which is suitable in planning-level comparisons of roundabouts with signalized intersections (Salamati et al., 2015). One example of using PEMS data was conducted in 2017 (Meneguzzer et al., 2017) with findings showing that roundabouts tended to reduce carbon dioxide (CO2) emissions after converting from signalized intersections, but signalized intersection had better performance in terms of nitrogen oxide (NOx).

Several studies also investigated the environmental impacts of roundabouts. The studies found that the emissions and fuel consumption were reduced after roundabout conversations (Hu et al., 2014; IIHS, 2022; Niittymäki, 1999; Várhelyi, 2002). For instance, the study on two double-lane roundabouts in Bellingham, Washington, estimated the fuel consumption and pollutant emissions (e.g., hydrocarbon (HC), Carbon monoxide (CO), CO2, and NOx) by using SIDRA. The results indicated that fuel consumption was reduced by 23 percent and vehicle emissions were reduced by 0 to 33 percent (Hu et al., 2014). However, one study discovered that the emissions was highly correlated to the drivers' behaviors and that the roundabout did not necessarily have a relationship with lower emissions compared to stop controlled and signalized intersections (Hallmark et al., 2011).

Lastly, roundabouts also show substantive benefits from an economic perspective, though the results of some studies are somewhat mixed. The 2011 MDOT study showed that average initial installation costs ranged from approximately $500,000 to nearly $2,000,000. Crash cost savings of $1.6M and $360K per year were estimated for single- and double-lane roundabouts, but triple-lane roundabouts showed a crash cost increase of $368,335 per year (Bagdade et al., 2011). The Maryland State Highway Administration also found that the crash cost saved $9.8M and the equivalent uniform annual cost was about $640,609 (Lawrence et al., 2015) resulting in a 15.3 benefit-cost ratio (BCR). A study from Nevada investigated the safety benefit-cost of a roundabout in detail (Wang, 2020) with higher BCR found at roundabouts with higher AADT and historical crashes (before the roundabout was built). Roundabouts cost less to operate than signalized intersections due to the lower initial and maintenance costs associated with signal installation, though costs are somewhat higher as compared to stop-controlled intersections. The FHWA report *Roundabout: An Information Guide* in 2000 indicated that the roundabouts save an average of

$5,000 per year in electricity and maintenance costs (Robinson et al., 2000). Additionally, the service life of a roundabout is about 25 years which is longer than the service life of signal equipment (i.e., 10 years). A Minnesota study showed that roundabouts have lower long-term costs than signalized intersections despite having similar construction costs at the same capacity (Leuer, 2017), while a Pennsylvania study estimated $36.4 million in savings based on a combination of societal economic costs from 14 roundabout conversion projects (Coffey et al., 2016).

2.4 Literature Summary

Generally, roundabouts show positive impacts on safety, operational, environmental, and economic performance due to unique geometric designs. Compared to a four-leg conventional intersection, a roundabout reduces the conflict point from 32 to 8, eliminating several severe crash types, such as head-on and right-angle crashes. The operation of roundabouts in terms of delay and capacity also performs better than conventional intersections. Additionally, roundabouts tend to provide extensive environmental and economic benefits as well. However, some studies have indicated that the total number of crashes increased after replacing conventional intersections with roundabouts, although the number of injury/fatal crashes decreased. Also, some studies showed that roundabouts only saw increased safely and efficiency over other intersection types under certain conditions (e.g., certain AADT or number of circulating lanes). Lastly, the influence of roundabouts on the pedestrian crossing and bicycle behaviors is under studied in the United States, and the current literature is rather barren when it comes to this topic.

This project investigated the performance of roundabouts in Michigan regarding driver behavior at roundabout entry, safety, operations, environmental benefits, and cost. Extensive field data collection activities were undertaken to collect driver behavior regarding speed, yielding, and gap acceptance at roundabout entry to investigate driver behavior. Subsequent safety analyses were also carried out at various levels of detail including naïve before-after, EB analysis, and cross-sectional analysis. SPFs for roundabouts were also developed based on number of approach legs and number of circulating lanes. Lastly, the benefits of roundabouts in terms of reduced delay and fuel savings were also determined. The results will be compared to the 2011 MDOT report and used as potential revisions to *MDOT Roundabout Design Aid*.

3 OPERATIONAL PERFORMANCE OF ROUNDABOUTS

Roundabouts have been shown to improve operational performance of intersections by reducing delay, crashes, and injuries, and improving gap acceptance and yielding behavior of vehicles. To that end, a series of operational data were collected at a subset of roundabout locations to assess several parameters including gap behavior, yielding behavior, and speed selection, that are critical to roundabout operations and safety. The following sections detail the field data collection procedure and the analysis results of operational behavior analysis at roundabouts.

3.1 Field Data Collection

3.1.1 Site Selection

In order to understand driver behavior at roundabouts, it was imperative to collect high-fidelity data including driver speed selection while entering a roundabout, their gap acceptance behavior, and their yielding behavior towards other vehicles in the circle and also to any pedestrian crossing the roundabout. To that end, a screening of all 180 roundabout locations in Michigan was conducted to identify sites that were suitable for field data collection. The screening process was done such that the candidate sites provided a diverse range of geometric characteristics, including number of approach legs (3 vs. 4), number of circulating lanes (1, 2, or 3), context (interchange vs. non-interchange, urban/rural/suburban), traffic volume ranges, and utilization by pedestrians and bicyclists. The relative safety performance between similar types of roundabouts were also considered in order to identify sites that are experiencing significantly more (or fewer) crashes than other, similar locations.

Based on the screening process and the recommendations from the research advisory panel (RAP), a total of 16 roundabout sites were identified that were finalized for field data collection, which are listed in Table 1. Out of these 16 sites, two sites- I-75 at Monroe M-46, and I-75 at M-81, had two roundabouts adjacent to each other with similar geometric design. The field data were collected at both of these roundabouts separately, thereby expanding total number of sites to 18. Six sites had facilities for pedestrian crossing including PHB at two sites. Seven sites out of 16 were located on interchange (exit ramps). One point to note is that some roundabouts are marked as having three-four or two-four legs in Table 1. These are the sites where the number of approaches entering the roundabout are not same as the total number of legs at the roundabout. For example, Figure 3 shows the roundabout located on exit ramp from southbound I-75 at M-81. There are only 3 total approaches from where the traffic enters the roundabout, but the roundabout has a total of four legs. Such cases are coded separately in the data as three-four (three entering approaches with four total legs) or two-four (two entering approaches with four total legs).

Table 1 List of Roundabouts Finalized for Field Data Collection

Site ID	Site	Type	Number of Legs	Context	Pedestrian Activity
1	M-5 at Pontiac Trail	Multilane	Four	Urban	None
2a	NB I-75 at Monroe M-46	Multilane	Three	Urban	None
2b	SB I-75 at Monroe M-46	Multilane	Three	Urban	None
3a	NB I-75 at M-81	Single lane	Three-four	Rural	None
3b	SB I-75 at M-81	Single lane	Three-four	Rural	None
4	US-10 at M-30	Single lane	Three	Rural	None
5	US-127 BR at Mission Road	Single lane	Three	Rural	None
6	US-23 at Lee Road	Multilane	Three-four	Urban	None
7	EB I-94 at Sprinkle Road	Multilane	Three-four	Rural	None
8	WB I-94 at Sprinkle Road	Single lane	Four	Urban	None
9	US-10 BR/M-20 at Patrick Road	Single lane	Two-four	Urban	None
10	NB I-75 at Bristol Road	Multilane	Four	Urban	None
11	US-23 at Geddes Road	Single lane	Three-four	Urban	Pedestrian crossing
12	M-52 at Werkner Road	Single lane	Four	Rural	Pedestrian crossing
13	Farmington at Maple Road	Multilane	Four	Urban	PHB
14	Drake at Maple Road	Multilane	Four	Urban	PHB
15	Geddes at Earhart Road	Single lane	Four	Rural	Pedestrian crossing
16	M-53 at 26 Mile Road	Multilane	Four	Rural	Pedestrian crossing

Figure 3 Southbound I-75 at M-81 with Three Entering Legs

3.1.2 Data Collection and Extraction

Once the list of roundabout sites was finalized, field data collection at each of the locations was completed by the MSU team. Four types of behavioral data were collected at each of the sites by various methods. These are discussed in detail in the following sections.

3.1.2.1 Approach Speeds

Prior research has shown various geometric characteristics to affect drivers' speed selection behavior while entering a roundabout, including approach width, flare length, inscribed circle diameter, central island diameter, circulatory roadway width, departure width, and split width (Ali & Flannery, 2006). Thus, to investigate how drivers' speed selection behavior varies across different roundabout locations, high-fidelity speed data were collected using handheld LIDAR (Light Detection and Ranging). An unmarked vehicle was parked outside of the shoulder, between 350 and 1,250 ft upstream of the yield sign located at roundabout entry. Continuous speed measurements were taken of vehicles entering the roundabout. The data were collected only during clear weather and dry pavement conditions. The LIDAR guns utilized in this study were ProLaser III manufactured by Kustom Signals Inc. These devices are able to measure vehicular speed and distance three times per second with an accuracy of ±1 mph at a range of 6,000 ft. Wherever possible, speed data collection was done at two approaches, one major and one minor approach, at each of the sites. However, due to parking restrictions and other site-specific factors, no speed data were collected at two out of 18 sites- US-10 BR/M-20 at Patrick Road, and NB I-75 at Bristol Road. At each of the sites where speed data were collected, nearly 120 vehicles per approach were tracked to get continuous speed data including both passenger cars and heavy vehicles.

Table 2 shows the information of data collection setup for each of the sites. Note that the distances in this table were measured using the LIDAR guns. The roundabout warning signs present at these sites were combinations of several signs and plaques from the Manual on Uniform Traffic Control Devices (Federal Highway Administration, 2009). These signs and plaques include circular intersection symbol sign (W2-6), advisory speed plaque (W13-1P), 'ROUNDABOUT' plaque (W16-17P), and 'XX FT' plaque (W16-2aP) as shown in Figure 4. Figure 5 displays an example of roundabout warning sign (combination of several signs and plaques) from site 10 (northbound direction). Note that sites with n/a values for the distance from the yield sign to roundabout warning sign do not necessarily indicate there were no warning signs present. At some of the sites due to the constraint in LIDAR setup, data collectors had to be positioned downstream of the warning sign, which resulted in no speed being recorded at or upstream of the warning signs.

Table 2 LIDAR Data Collection Setup Information

Site ID	Site	Direction	Distance from Yield Sign to LIDAR (ft)	Distance from Yield Sign to Roundabout Warning Sign (ft)
1	M-5 at Pontiac Trail	Northbound	740	n/a
1	M-5 at Pontiac Trail	Westbound	1,145	945
2a	NB I-75 at Monroe M-46	Westbound	496	n/a
3a	NB I-75 at M-81	Westbound	1,010	665
3b	SB I-75 at M-81	Eastbound	1,010	740
4	US-10 at M-30	Southbound	545	n/a
5	US-127 BR at Mission Road	Southbound	407	n/a
6	US-23 at Lee Road	Southbound ramp	587	448
7	EB I-94 at Sprinkle Road	Northbound	927	n/a

Site ID	Site	Direction	Distance from Yield Sign to LIDAR (ft)	Distance from Yield Sign to Roundabout Warning Sign (ft)
8	WB I-94 at Sprinkle Road	Westbound ramp	645	n/a
11	US-23 at Geddes Road	Northbound	506	382
12	M-52 at Werkner Road	Northbound	772	524
12	M-52 at Werkner Road	Southbound	1,225	1,106
13	Farmington at Maple Road	Eastbound	386	n/a
14	Drake at Maple Road	Westbound	625	n/a
15	Geddes at Earhart Road	Eastbound	705	255
15	Geddes at Earhart Road	Southbound	360	n/a
16	M-53 at 26 Mile Road	Northbound ramp	355	n/a

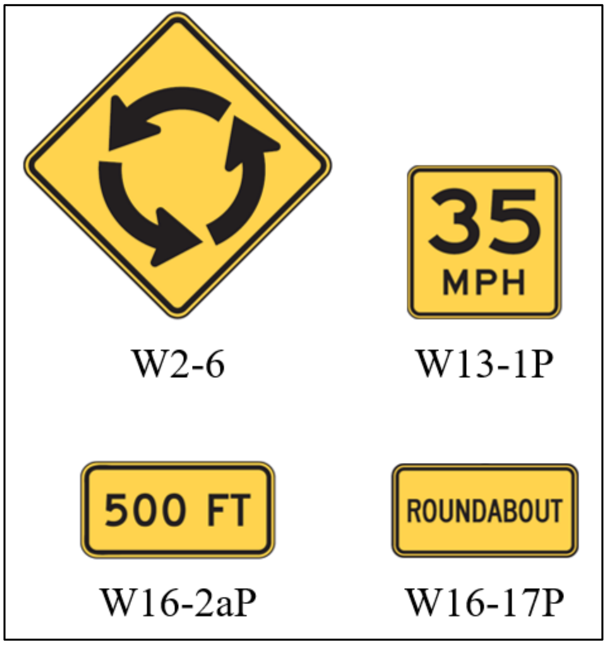

Figure 4 Roundabout Warning Signs and Plaques (Federal Highway Administration, 2009)

Figure 5 Roundabout Warning Sign at M-52 at Werkner Road (Site 12 Northbound Direction)

During the data collection process, LIDAR technicians would start tracking target vehicles as soon as they passed the LIDAR position. The speeds and ranges were continuously recorded until the yield sign at the roundabouts, where possible. In cases where a pedestrian was present, speeds and ranges were recorded until target vehicle stop (i.e., before the pedestrian crossing or at the yield sign in the events of fail to yield to pedestrian).

Each LIDAR gun was connected to a laptop, which allowed all speeds and ranges to be recorded in real-time using proprietary software. The information saved included timestamp, range, and speed for each target vehicle. The software also allowed remarks to be entered for each observation after completing LIDAR tracking. In this study, vehicle color and type were recorded, in addition to any other comments. In cases where a dummy pedestrian was present, yielding behavior towards pedestrian was recorded (i.e., yielded or not). For roundabouts with PHB, signal information when dummy pedestrian tried to cross the road was recorded (i.e., flashing yellow, solid red, and flashing red). Figure 6 shows raw data of speed profiles from site 4 (US-10 at M-30).

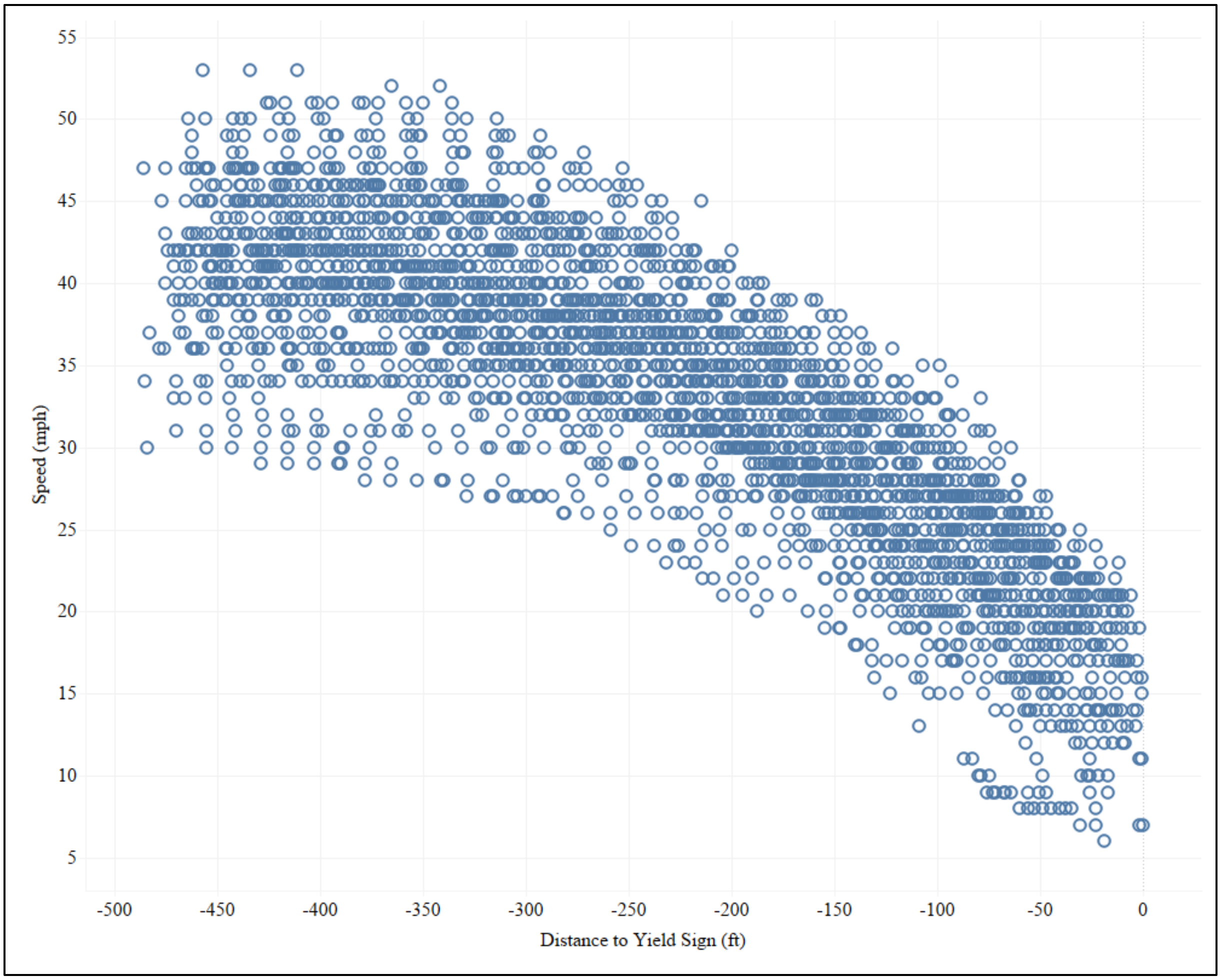

Figure 6 Raw Data of Speed Profile from Roundabout at US-10 at M-30 (Site 4 Southbound Direction)

One drawback of using LIDAR guns to obtain vehicle speed profile is that the speeds cannot be measured at the same locations along the road for each vehicle. Thus, it was needed to convert the data into a series of spot speeds to allow for evaluation at specific reference points. Consequently, linear interpolation method was used to obtain speed at every foot interval using the available adjacent speeds through RStudio Version 4.1.1. After obtaining the speed data at every foot interval, data were further reduced at every 50 ft interval, starting from the yield sign at the roundabout, moving upstream. Missing speed data at the first 150 ft from the yield sign were extrapolated. These missing data typically happened when vehicles stop/rolling stop before the yield signs. It also happened when the view of LIDAR technicians was blocked by other incoming vehicles. Figure 7 shows the speed profile for each vehicle at every 50 ft interval for site 3b (SB I-75 at M-81).

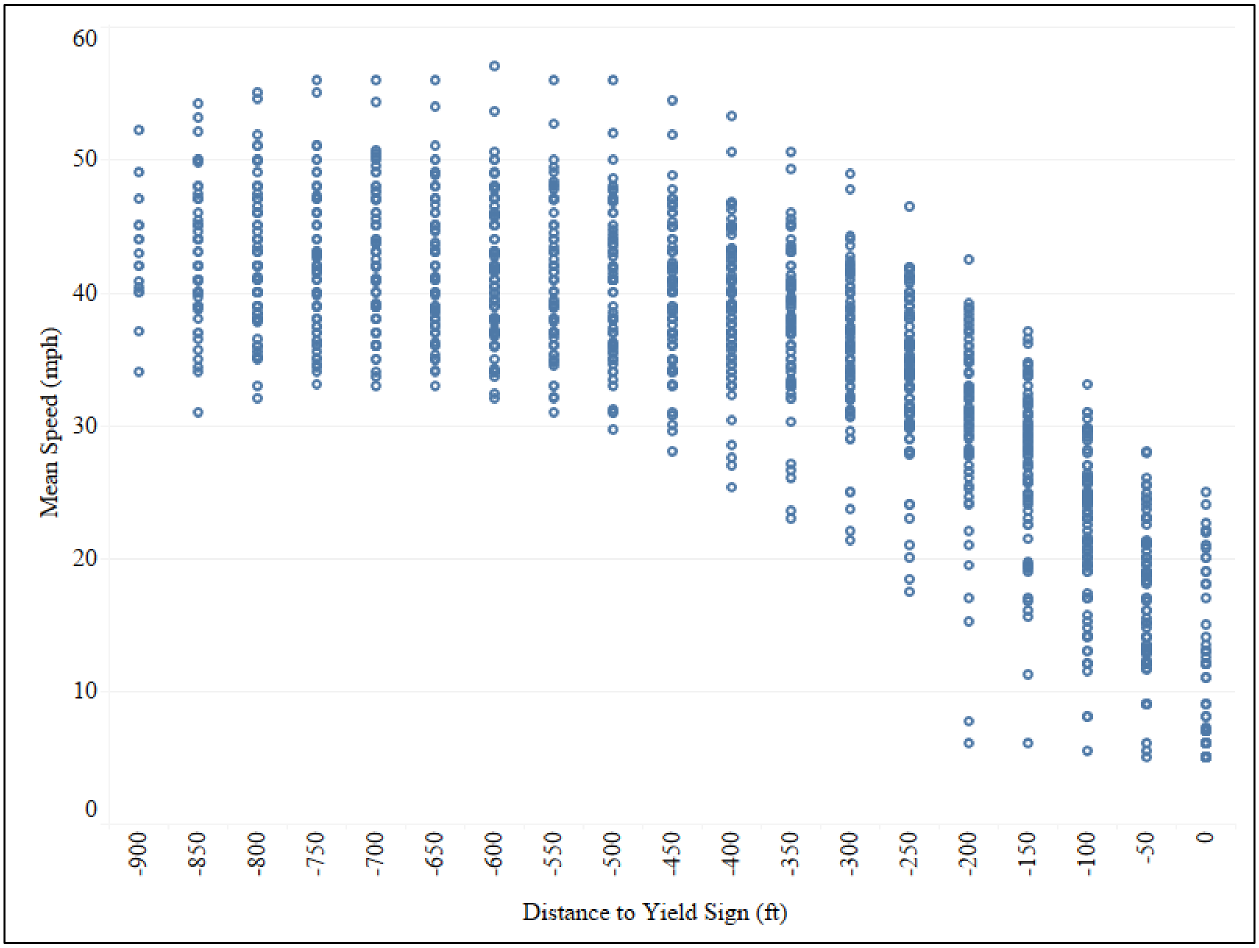

Figure 7 Interpolated Speed Profile at Every 50 ft Interval for Site 3b (SB I-75 at M-81)

3.1.2.2 Gap Acceptance and Rejection

Other driver behavior investigated was gap acceptance and gap rejection behavior at roundabouts. To that end, elevated cameras were installed at two approaches at each of the 18 sites finalized for field data collection that recorded the behavior of vehicles entering the roundabout and their interaction with traffic in the circle. At site IDs 2b, 8, 9, and 14, video data were collected at only one of the approaches due to equipment failure or inability to collect video data due to site restrictions such as the presence of a work zone. Thus, in total, video data were collected at 32 approaches across 18 sites for 2 to 3 hours at each approach. Again, video data were recorded during clear weather and dry pavement conditions only.

Once the data collection was completed, the recordings were manually reviewed in the MSU labs to extract relevant information related to gap acceptance behavior. Free-flowing vehicles entering the roundabout in between two vehicles in the traffic circle were identified. Information such as vehicle type and lane position were recorded. Additionally, two important timestamps were also recorded that allowed the determination of rejected gaps and accepted gap for each of the subject vehicles. To determine these gaps, two reference points were marked in every video. Figure 8 shows these reference points, A and B (Belz & Yang, 2018). The first timestamp was recorded when the subject vehicle entering the roundabout reached point B, i.e., entry point of roundabout. This point marked the time when the subject vehicle began the process of merging into the roundabout and was then followed by successful entry into the gap in the circle. The second timestamp was recorded when the vehicles in the circle reached point A. These two timestamps allowed for the determination of both rejected gaps and the accepted gap by the subject vehicle.

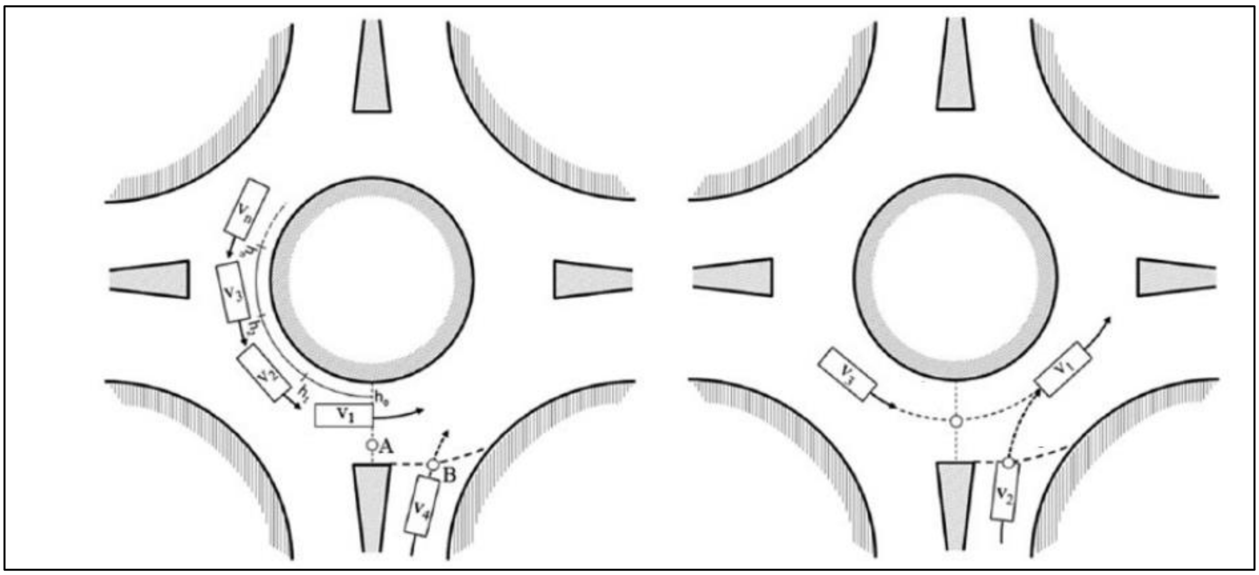

Figure 8 Reference Points for Video Data Extraction for Gap Acceptance Behavior

The accepted gap was determined as the time gap between when the subject vehicle reached point B, enters the traffic circle, followed by a vehicle in the circle reaching point A. Only accepted gaps less than 9 seconds were considered for analysis purposes (Belz & Yang, 2018). Cases where the subject vehicle and the next vehicle from the left in the circle arrived at their respective reference points at the same time were also ignored since they generated an accepted gap of zero seconds, although the sample size of such cases was negligible (only 7 out of more than 2,000 accepted gaps extracted). If the subject vehicle arrived at the entry point, but had to wait for a gap to merge into the circle, rejected gaps were calculated by taking the time gap between consecutive vehicles in the circle as they arrive at point A, i.e., difference in timestamp when a vehicle in the circle arrived at point A and the previous vehicle in the circle that arrived at point A. For rejected gaps, a maximum of five rejected gaps were recorded for each subject vehicle to increase the efficiency of data extraction from the videos. An example of rejected gap is shown in Figure 9.

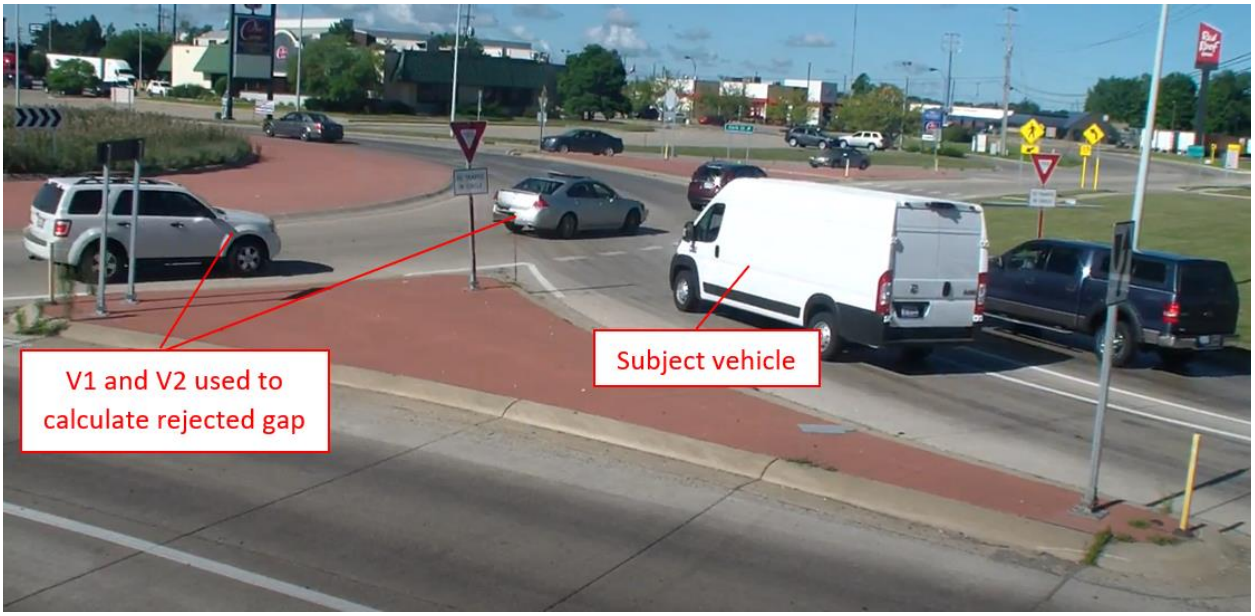

Figure 9 An Example of Rejected Gap at Roundabout on WB I-94 at Sprinkle Road

3.1.2.3 Yielding Behavior at Roundabout Entry

Drivers approaching a roundabout are required to yield to vehicles already in the traffic circle and wait for a gap in the circle prior to merging. To investigate driver yielding behavior towards vehicles in the traffic circle, video data collected at each of the sites were utilized. First, vehicles attempting to enter the roundabout when a vehicle was already present in the traffic circle on the left side of the subject vehicle were identified. Thereafter, the yielding behavior of the subject vehicle was recorded in the form of a binary variable which was coded as 1 if the subject vehicle yielded to traffic in circle, and 0 if it did not yield. Additional details about subject vehicle behavior such as whether the subject vehicle simply slowed down to yield or it came to a complete stop, or whether the vehicle simply proceeded through the roundabout without slowing down. In cases where the subject vehicle stopped to yield to traffic in circle, the location where the vehicle stopped, i.e., after the yield line, or at or before the yield line, was also recorded.

3.1.2.4 Yielding Behavior to Pedestrians

Six sites finalized for field data collection had facilities for pedestrian crossings on at least one of their approaches. Two of the sites had PHB installed on all of their approaches. Thus, it was decided to examine driver yielding behavior towards pedestrians utilizing these crossings. However, the pedestrian activity at these sites was too low to collect any meaningful data based on natural pedestrian crossing events. Therefore, pedestrian crossing events were staged at each of the sites by MSU research team to simulate actual pedestrian crossing events. The following procedure was adopted to stage pedestrian crossing events that simulate real-life crossing scenarios at each of the sites:

1 A staged pedestrian stood on the roadside near the crosswalk entrance and waited for a vehicle to approach the crosswalk.

2 When a vehicle approached the crossing, the pedestrian indicated their intention to cross by standing at the curb with one foot in the crosswalk while facing the oncoming traffic. The distance of the pedestrian from the vehicle when they first showed their intention to cross varied from 100 ft to 150 ft based upon site-specific factors such as approach speeds and stopping sight distance.

3 The staged pedestrian started crossing the road when the driver in the nearest lane yielded, and maintained eye contact with the driver at all times.

4 If there were additional vehicles approaching in other lanes and the staged pedestrian had already crossed halfway, the staged pedestrian waited until the intention of the approaching vehicle was determined.

5 The procedure was repeated until a target sample size was obtained (50 crossings per entrance).

Figure 10 shows an example of how the staged pedestrian crossing events were carried out. The figure shows a pedestrian showing his intention to cross the crosswalk when the vehicle was 150 ft from the crosswalk. Video cameras were installed at each of these locations to record all the staged pedestrian crossing events.

Figure 10 Staged Pedestrian Crossing Event at Roundabout Crossings

In cases where traffic control devices were present to facilitate pedestrian crossings, such as PHB, similar crossing events were staged. The pedestrian indicated his/her intention to cross by pressing the pedestrian button and activating the signal. The video camera recorded the time when the pedestrian activated the signal, and whether the driver yielded to the pedestrian or not. The video data also helped in recording driver behavior during different phases of the signal such as solid red and flashing red.

Each of the sites also had pedestrian crossing warning signs installed. Figure 11 shows the different types of pedestrian warning signs installed across the five sites. The in-street warning sign was installed at sites 13, 14, and 15. Site 11 had the yield here to pedestrian sign, while site 16 had the standard yellow diamond sign.

(a) Standard yellow diamond (M-53 at 26 Mile Road)

(b) Yield here to ped sign (US-23 at Geddes Road)

(c) In-street warning sign (Geddes at Earhart Road)

Figure 11 Types of Pedestrian Crossing Warning Signs

3.1.3 Data Integration and Preparation

The field data collected were extracted and cleaned as discussed previously. For analysis purposes, four separate datasets were prepared. This included datasets related to driver speed selection, gap behavior, vehicle-to-vehicle yielding behavior, and vehicle-to-pedestrian yielding behavior. Each of the four datasets were integrated with site information and site geometric characteristics pertinent to the approach on which the data were collected. This information was collected manually using street view on Google Earth and during field data collection. This included several variables as listed below:

- Presence and type of roundabout warning sign (no sign, warning sign with or without beacon or LED light)
- Advisory speed limit
- Speed limit of the approach
- Presence and type of pedestrian crossing sign
- Presence of refuge island
- Number of approach lanes
- Presence of exclusive right-turn lanes
- Roundabout geometry characteristics such as number of circulatory lanes, number of legs, maximum diameter of inscribed circle
- Whether the roundabout is located on freeway exit ramp
- Roadway context (rural or urban)
- Traffic volume entering the roundabout using the approach of interest

3.2 Data Analysis and Results

The following sections discuss the methodology for data analysis and present the analysis results separately for each of the four datasets.

3.2.1 Driver Speed Selection

The objective of this analysis was to investigate driver's speed selection behavior as they approach a roundabout. Roundabouts are constructed with the intention of forcing the drivers to slow down near the entry and yield to traffic in the circle. To that end, LIDAR speed data collected at each of the sites was reduced as discussed previously in Section 3.1.2.1. The speed data were integrated with respective roundabout site geometry including number of legs, number of lanes, speed limit at approach, roadway context, diameter of central island of the roundabout, number of approach lanes, whether the roundabout is located on an interchange or not, AADT, etc. Table 3 presents the general overview of the sites included in this analysis. LIDAR speed data were collected at two approaches at two of the sites. Thus, the data includes total 16 approaches across 14 sites. The

AADT for these 16 approaches had an average value of 8,588 vehicles/day (sd = 4,686). It was also observed that six of 16 approaches had pedestrian crossings and seven sites were in urban areas (with nine approaches in rural areas). The average center island diameter was 129 ft (min = 82; max = 196; sd = 43), with the roundabout warning sign present on every approach. Additional traffic control devices were observed on several sites such as flashing beacons which were present on two of the total 16 approaches. PHBs were also present at two sites (sites 13 and 14).

As alluded to previously, speed was measured from the furthest distance from where safe data collection was possible. Each vehicle was tracked until it reached the roundabout (yield sign or yield line). The furthest distance was 1,000 ft upstream of the roundabout while the shortest distance was 250 ft upstream of the roundabout. On average, at every site 127 vehicles were observed (min = 80; max = 167; sd = 20).

For analysis purposes, indicator variables were created for distance from roundabout yield line. Several other variables of interest were also converted into binary indicators. For example, the type of vehicle was divided into two categories- passenger cars or heavy vehicles. The posted speed limit on the approach was also divided into three groups: speed limit less than or equal to 40 mph (includes speed limit of 35 mph and 40 mph), speed limit from 45 mph to 55 mph, and approaches on ramps exiting from freeways posted at 70 mph.

Table 3 General Overview of Sites for Speed Data Analysis

Site	Direction	Name	Approach Name	Lanes	Circulating lane	Legs	Advisory Speed [mph]	Speed Limit [mph]	AADT
1	NB	M-5 at Pontiac Trail	M-5	2	3	4	20	55	17,381
1	WB	M-5 at Pontiac Trail	Pontiac Trail	3	3	4	20	45	13,931
2a	WB	NB I-75 at Monroe (M-46)	Holland Rd	2	2	3	20	45	6,948
3a	WB	NB I-75 at M-81	Washington Rd	1	1	3/4	20	45	7,752
3b	EB	SB I-75 at M-81	Washington Rd	1	1	3/4	20	45	4,115
4	SB	US-10 at M-30	M-30	1	1	3	20	55	2,671
5	SB	US-127 BR at Mission Road	US-127 Mission Rd	1	1	3	20	45	3,793
6	SB	SB US-23 at Lee Road	US-23 Exit Ramp	3	2	3/4	20	60	7,407
7	NB	EB I-94 at Sprinkle Road	Sprinkle Rd	2	2	3/4	15	45	14,228
8	WB	WB I-94 at Sprinkle Road	Sprinkle Rd	2	1	4	15	45	15,704
11	NB	NB US-23 at Geddes Road	US-23 Exit Ramp	2	1	3/4	20	60	5,960
13	EB	Farmington at Maple	Maple Rd	3	2	4	20	45	11,161
14	WB	Drake at Maple	Maple Rd	3	2	4	20	45	10,666
15	EB	Geddes at Earhart	Geddes Rd	1	1	4	20	40	6,467
15	SB	Geddes at Earhart	Earhart Rd	1	1	4	20	35	3,128
16	NB	NB M-53 at 26 Mile Road	M-53 Exit Ramp	2	2	4	No Advisory Speed	60	6,101

3.2.1.1 Statistical Analysis

Given the interrelationships between the factors impacting speeds as vehicles approach the yield line, random effects linear regression models were estimated to discern those factors that are associated with driver speed selection. The general form of the model is shown below:

$$Y_i = \beta_o + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_k X_k + u_i + \varepsilon_i \quad \text{Equation 8}$$

Where,

Y_i = Speed at site i,

X = Vector of parameters that influence driver's speed selection behavior such as traffic volume, number of lanes, etc.,

β = Vector of estimable parameters that quantify the effects of these parameters,

u = Random effect that captures unobserved vehicle-specific effects and remains same for a given vehicle, and

ε = error term.

Two separate models were developed- one when there is no pedestrian present at the crosswalk, and other when a pedestrian is waiting at the curb to cross the roundabout. This is because if a pedestrian is present, then drivers may yield to the pedestrian, thereby restricting the distance to which the speed data can be collected till the crosswalk or pedestrian stop line, if present. Six out of 16 approaches had a pedestrian crossing and were included in the second model. Two of these six sites (site 13 and 14) had a pedestrian stop line while the remaining 4 sites only had a crosswalk with no stop line. It should be noted that the same two sites also had PHB facility. Average distance between pedestrian stop line (or crosswalk) and the roundabout yield line was 58 ft.

Table 4 and Table 5 show the results of the estimated model, respectively, in the absence and presence of a pedestrian. The table shows estimated coefficients along with standard error of these estimates, t-statistic, and p-value. A total of 17,803 speed measurements were included in the no-pedestrian case, while 1,542 speed measurements were analyzed in the presence of pedestrian.

Table 4 Random Effects Linear Regression for Speed at Roundabout Approach in Absence of Pedestrian

Parameter	Estimate	Std. Error	t-stat	p-value
Intercept	41.596	0.335	124.22	<0.001
Vehicle yielded to traffic in circle (1 if yes)	-3.261	0.333	-9.788	<0.001
AADT per thousand	0.132	0.025	5.377	<0.001
Distance from yield line (ft)				
600+		*Baseline*		
550	-0.913	0.139	-6.548	<0.001
500	-1.114	0.127	-8.801	<0.001
450	-1.495	0.118	-12.700	<0.001
400	-2.287	0.114	-20.160	<0.001
350	-3.596	0.113	-31.833	<0.001
300	-5.850	0.108	-54.372	<0.001
250	-7.877	0.105	-74.906	<0.001
200	-10.447	0.106	-98.701	<0.001
150	-14.011	0.107	-131.320	<0.001
100	-18.761	0.109	-172.932	<0.001
50	-24.994	0.110	-227.015	<0.001
0	-29.745	0.118	-251.521	<0.001
Vehicle type				
Passenger car		*Baseline*		
Heavy vehicle	-7.154	0.395	-18.136	<0.001
Posted speed limit (mph)				
35-40		*Baseline*		
45-55	2.876	0.358	8.044	<0.001
Exiting from freeway posted at 70 mph	4.263	0.442	9.640	<0.001
Approach lane width (ft)				
> 12		*Baseline*		
<= 12	-2.148	0.286	-7.511	<0.001
Right turn bypass lane				
Not present		*Baseline*		
Present	-0.965	0.267	-3.618	<0.001
Variance of Intercept	16.397	0.591	27.725	<0.001

In the absence of pedestrian, the results showed that speeds were significantly lower among vehicles that yielded to cross road traffic. Speeds were higher among non-yielding vehicles, which was generally reflective of gaps in circulating traffic that allowed for such higher speeds. Sites with higher AADT exhibited higher speeds, which may reflect other differences at these locations (e.g., higher numbers of lanes, larger entry radii). Vehicle speeds were consistently reduced as vehicles approached the yield line. For example, these reductions were most pronounced when vehicles were within 200 ft of the yield line. The reduction in speeds was only 2.3 mph at a distance 400 ft upstream of the yield line, but 10.4 mph at a distance 200 ft upstream, when compared to speeds 600 ft or more upstream of yield line. As expected, truck speeds were significantly lower than those of passenger cars. Speeds on sites located on exit ramps were also significantly higher than sites not on interchanges. Lastly, speeds were also affected by other site-specific geometric characteristics such as approach lane widths and presence of right turn bypass lanes.

Table 5 Random Effects Linear Regression for Speed at Roundabout Approach in Presence of Pedestrian

Parameter	Estimate	Std. Error	t-stat	p-value
Intercept	39.489	0.624	63.322	<0.001
Vehicle yielded to pedestrian (1 if yes)	0.466	0.590	0.790	0.430
Distance from pedestrian stop line (ft)				
400+		*Baseline*		
300-400	-0.387	0.360	-1.075	0.283
250-300	-1.566	0.398	-3.941	<0.001
200-250	-3.401	0.372	-9.143	<0.001
150-200	-6.101	0.363	-16.817	<0.001
100-150	-9.872	0.363	-27.175	<0.001
50-100	-15.898	0.365	-43.568	<0.001
0-50	-23.889	0.385	-62.053	<0.001
Vehicle type				
Passenger car		*Baseline*		
Heavy vehicle	-5.845	1.625	-3.597	<0.001
Roundabout type				
Single lane		*Baseline*		
Multilane	-2.099	0.578	-3.630	<0.001
Roundabout location				
Surface street		*Baseline*		
Exit ramp	4.183	0.593	7.050	<0.001
Variance of Intercept	18.552	1.819	10.199	<0.001

When the pedestrian was present at roundabout crossing, the results tended to be similar. However, surprisingly, there were no significant differences in speeds between vehicles that yielded to pedestrian and those who did not. In terms of distance, reduction in speeds was more pronounced when vehicle was 50-100 ft from pedestrian crosswalk or stop line. Again, heavy vehicles had lower speeds compared to passenger cars. In terms of roundabout geometry, multilane roundabouts exhibited lower speeds than single lane roundabouts, and roundabouts located on exit had higher average speeds than roundabouts on surface streets.

3.2.2 Gap Behavior

The accepted gaps and rejected gaps were extracted from the videos at 32 approaches across 18 sites as discussed previously. For each subject vehicle entering the roundabout in between two vehicles, one accepted gap and a maximum of five rejected gaps were calculated (accepted gaps of 0 seconds or gaps larger than 9 seconds were ignored). Table 6 and Table 7 present the descriptive statistics of accepted gaps by site for all vehicles combined, and when both the subject vehicle and the vehicle in the circle were passenger cars, respectively. Some of the sites, such as site 2a, 2b, 5, 9, etc., had limited sample sizes (i.e., less than 15 vehicles) due to low traffic volumes, which resulted in fewer vehicles entering the roundabout between consecutive vehicles with accepted gaps less than 9 seconds. Similarly, Table 8 and Table 9 present the descriptive statistics of rejected gaps by site for all vehicles combined and for passenger cars only, respectively.

The results showed that the mean accepted gaps ranged from 3.3 s to 6.9 s for passenger cars across all sites. However, there were few sites where only a few accepted gaps were possible to extract from the video (less than or equal to 15) due to reasons alluded to previously. However, considerable variation was found in mean accepted gap from site-to-site. Similarly, the rejected gap for passenger cars varied from 1.4 s to 3.5 s. When averaged across all sites, the rejected gap for both passenger cars and all vehicles combined was 2.2 s which is largely due to small sample size of heavy vehicles compared to passenger cars. Similarly mean accepted gap across all sites was 5.3 s for both passenger cars and all vehicles combined.

Table 6 Accepted Gaps by Site for All Vehicles Combined

Site ID	Site	Sample Size	Min	Max	Mean	Std. Dev.
1	M-5 at Pontiac Trail	116	0.01	8.73	4.82	2.18
2a	NB I-75 at Monroe M-46	14	3.62	8.79	6.35	1.53
2b	SB I-75 at Monroe M-46	4	0.37	7.29	3.34	3.09

Site ID	Site	Sample Size	Min	Max	Mean	Std. Dev.
3a	NB I-75 at M-81	45	2.10	8.99	6.41	1.93
3b	SB I-75 at M-81	85	0.01	8.98	5.78	2.12
4	US-10 at M-30	24	0.08	8.46	3.85	3.15
5	US-127 BR at Mission Road	4	1.42	6.24	4.28	2.04
6	US-23 at Lee Road	81	1.79	8.93	5.65	1.59
7	EB I-94 at Sprinkle Road	95	1.08	8.99	5.56	2.00
8	WB I-94 at Sprinkle Road	33	4.70	8.64	6.80	1.15
9	US-10 BR/M-20 at Patrick Road	14	1.60	8.74	5.79	2.00
10	NB I-75 at Bristol Road	82	0.02	8.93	4.85	2.29
11	US-23 at Geddes Road	109	0.28	8.93	5.66	1.97
12	M-52 at Werkner Road	17	1.10	8.99	6.10	2.05
13	Farmington at Maple Road	129	0.01	8.69	5.19	1.80
14	Drake at Maple Road	47	0.27	8.88	4.19	2.08
15	Geddes at Earhart Road	15	0.35	8.22	5.22	1.94
16	M-53 at 26 Mile Road	104	0.38	8.71	5.44	1.92

Table 7 Accepted Gaps by Site for Passenger Cars

Site ID	Site	Sample Size	Min	Max	Mean	Std. Dev.
1	M-5 at Pontiac Trail	102	0.01	8.73	5.01	2.14
2a	NB I-75 at Monroe M-46	11	4.85	8.79	6.32	1.34
2b	SB I-75 at Monroe M-46	4	0.37	7.29	3.34	3.09
3a	NB I-75 at M-81	20	2.67	8.99	6.88	1.63
3b	SB I-75 at M-81	51	2.12	8.98	6.03	1.78
4	US-10 at M-30	21	0.08	8.46	3.90	3.19
5	US-127 BR at Mission Road	4	1.42	6.24	4.28	2.04
6	US-23 at Lee Road	74	1.79	8.93	5.60	1.63
7	EB I-94 at Sprinkle Road	79	1.08	8.99	5.37	1.92
8	WB I-94 at Sprinkle Road	20	4.79	8.49	6.68	1.10
9	US-10 BR/M-20 at Patrick Road	13	1.60	8.74	5.96	1.98
10	NB I-75 at Bristol Road	52	0.02	8.93	4.81	2.25
11	US-23 at Geddes Road	104	0.28	8.93	5.65	2.01
12	M-52 at Werkner Road	13	1.10	8.99	6.13	2.17
13	Farmington at Maple Road	125	0.01	8.69	5.22	1.80
14	Drake at Maple Road	42	0.27	8.44	4.00	1.88
15	Geddes at Earhart Road	15	0.35	8.22	5.22	1.94
16	M-53 at 26 Mile Road	100	0.38	8.71	5.39	1.93

Table 8 Rejected Gaps by Site for All Vehicles Combined

Site ID	Site	Sample Size	Min	Max	Mean	Std. Dev.
1	M-5 at Pontiac Trail	366	0.02	5.82	1.66	1.05
2a	NB I-75 at Monroe M-46	49	0.02	6.91	2.26	1.74
2b	SB I-75 at Monroe M-46	24	0.12	8.19	3.35	2.09
3a	NB I-75 at M-81	135	0.08	8.43	3.12	1.83
3b	SB I-75 at M-81	218	0.02	9.78	3.41	1.85
4	US-10 at M-30	45	0.07	8.68	2.00	1.80
5	US-127 BR at Mission Road	6	0.07	11.68	3.00	4.49
6	US-23 at Lee Road	293	0.02	8.07	1.79	1.31
7	EB I-94 at Sprinkle Road	362	0.05	8.58	2.07	1.48
8	WB I-94 at Sprinkle Road	126	0.03	8.07	1.68	1.77
9	US-10 BR/M-20 at Patrick Road	57	0.05	5.76	1.78	1.44
10	NB I-75 at Bristol Road	271	0.18	8.17	2.20	1.34
11	US-23 at Geddes Road	188	0.02	6.14	2.56	1.08
12	M-52 at Werkner Road	78	0.05	9.33	1.59	1.67
13	Farmington at Maple Road	331	0.03	9.38	1.75	1.31
14	Drake at Maple Road	116	0.07	4.25	1.46	0.95
15	Geddes at Earhart Road	62	0.03	4.32	1.74	1.14
16	M-53 at 26 Mile Road	320	0.03	10.24	1.58	1.30

Table 9 Rejected Gaps by Site for Passenger Cars

Site ID	Site	Sample Size	Min	Max	Mean	Std. Dev.
1	M-5 at Pontiac Trail	329	0.02	5.82	1.64	1.05
2a	NB I-75 at Monroe M-46	45	0.02	6.91	2.23	1.73
2b	SB I-75 at Monroe M-46	22	0.57	8.19	3.53	2.06
3a	NB I-75 at M-81	53	0.08	8.27	3.29	1.91
3b	SB I-75 at M-81	140	0.02	9.78	3.30	1.85
4	US-10 at M-30	33	0.07	8.68	2.08	1.76
5	US-127 BR at Mission Road	6	0.07	11.68	3.00	4.49
6	US-23 at Lee Road	264	0.02	8.07	1.82	1.33
7	EB I-94 at Sprinkle Road	289	0.05	8.58	2.08	1.44
8	WB I-94 at Sprinkle Road	88	0.03	8.07	1.85	1.92
9	US-10 BR/M-20 at Patrick Road	56	0.05	5.76	1.77	1.46
10	NB I-75 at Bristol Road	224	0.18	8.17	2.15	1.29
11	US-23 at Geddes Road	175	0.02	6.14	2.58	1.10
12	M-52 at Werkner Road	65	0.05	9.33	1.57	1.61
13	Farmington at Maple Road	316	0.03	9.38	1.73	1.31
14	Drake at Maple Road	98	0.07	3.65	1.37	0.86
15	Geddes at Earhart Road	61	0.03	4.32	1.76	1.14
16	M-53 at 26 Mile Road	296	0.03	10.24	1.59	1.31

Both accepted gap and rejected gap depend on driver characteristics, roundabout geometry, and site-specific characteristics such as traffic volume and roadway context. To that end, the probability of accepting and rejecting a gap presented to the driver entering the roundabout is calculated based on the site characteristics. For example, Figure 12 presents the cumulative probability of accepting or rejecting a gap separately for single-lane and multilane roundabouts for passenger cars only. The figure can also be used to determine the critical gap for the two roundabout types. Critical gap is defined as the smallest gap that drivers are willing to accept (Troutbeck, 2016). In Figure 12, the point of intersection of the cumulative probabilities of accepting and rejecting gaps is the critical gap. At single-lane and multilane roundabouts, the critical gap is 3.9 s and 3.1 s, respectively. This shows that drivers tend to accept smaller gaps at multilane roundabouts compared to single-lane roundabouts.

This method of determining critical gap is similar to Raff's method, although the present analysis considers all rejected gaps instead of considering just the maximum rejected gap. Similar probability distributions were plotted by grouping sites based on their geometry. Figure 13, Figure

14, and Figure 15 show similar plots when roundabouts are grouped based on number of approach legs, roadway context, and based on whether the roundabout is located on ramp terminals or not, respectively. The figures also help in understanding how site characteristics influence driver's tendency to accept or reject a gap. For example, drivers are more likely to accept shorter gaps in urban areas compared to in rural areas. This may be a function of traffic volume and number of circulatory lanes. Similarly, drivers tend to accept larger gaps at roundabouts on ramp terminals compared to roundabouts on surface roads. Intuitively, accepted gaps should be smaller on roundabouts at ramp terminals due to higher speeds of vehicles on ramps, however, traffic volume on the cross road largely influences the gap accepted. Additionally, the traffic coming from the left may have to navigate through another controlling feature such as traffic signal or roundabout (at the location of exit ramp for other direction) which may lead to traffic arriving at the roundabout in platoon, thereby increasing the accepted gap. Table 10 summarizes the plots and presents critical gaps estimated for each of the group of roundabout types.

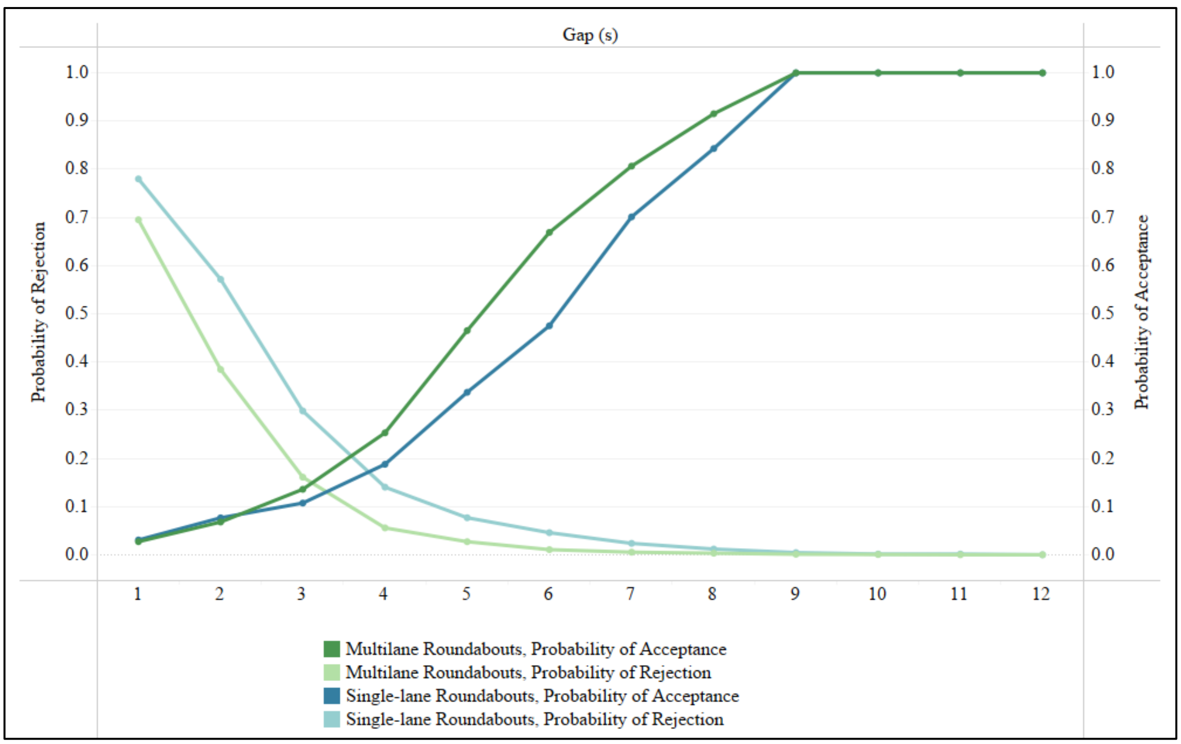

Figure 12 Critical Gap at Single-lane and Multi-lane Roundabouts

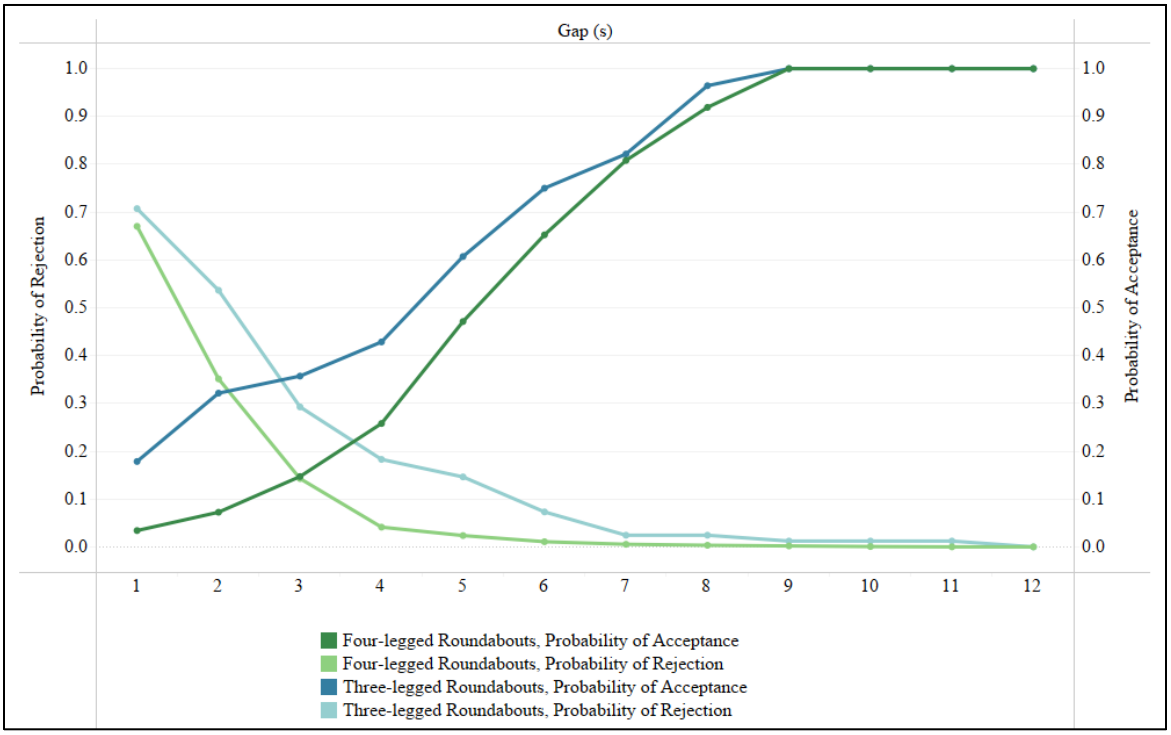

Figure 13 Critical Gap at Three-Legged and Four-Legged Roundabouts

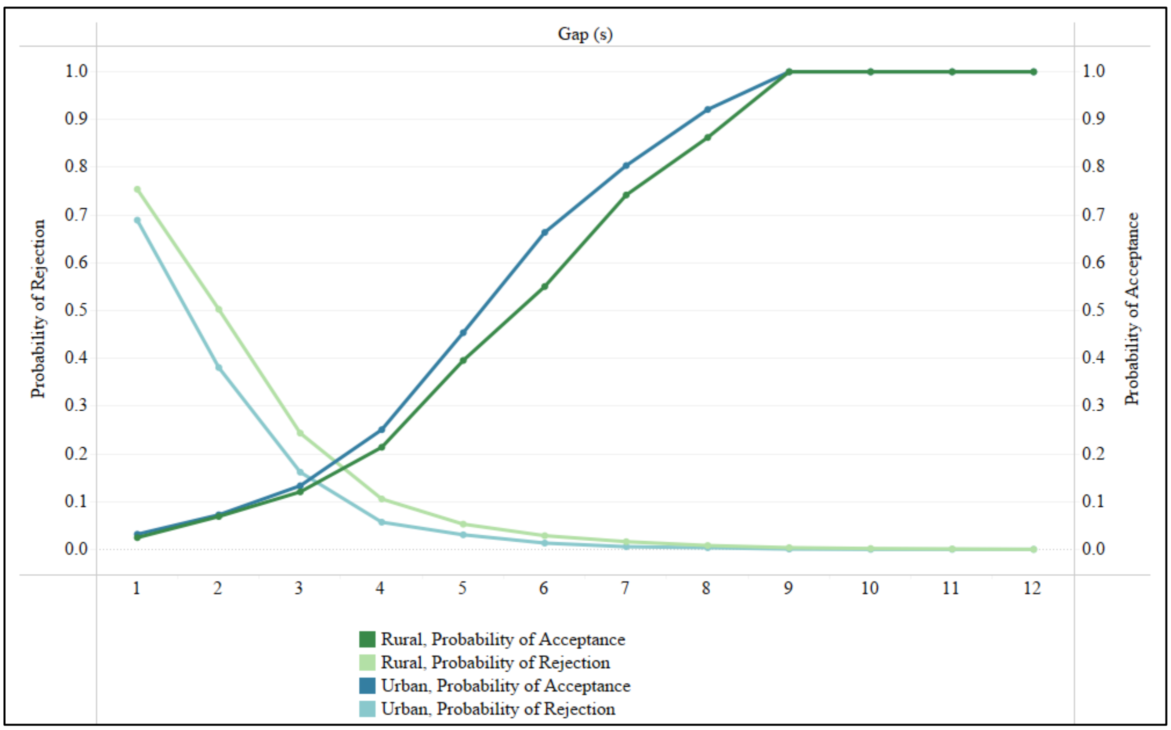

Figure 14 Critical Gap at Urban and Rural Roundabouts based on Roadway Context

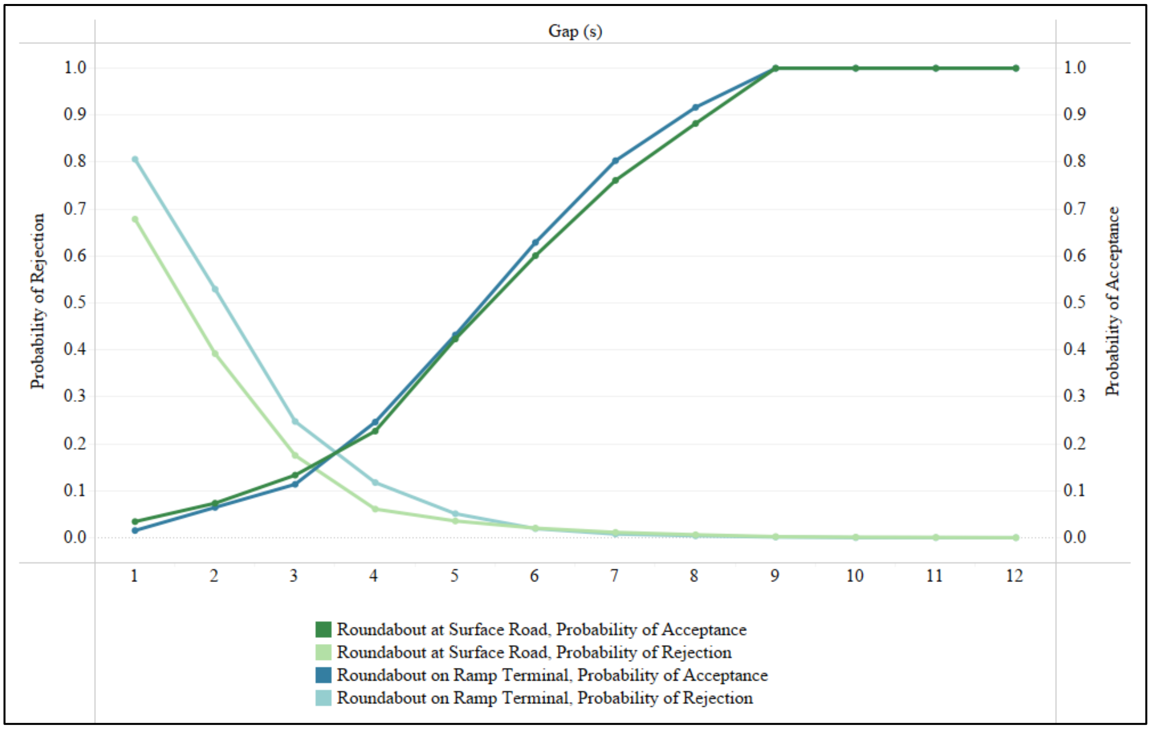

Figure 15 Critical Gap at Roundabouts based on Location (Surface Street vs Ramp Terminal)

Table 10 Critical Gap based on Roundabout Category

Roundabout Category	Critical Gap (s)
Single lane	3.9
Multilane	3.1
Three-legged	2.8
Four-legged	3.0
Rural context	3.1
Urban context	3.5
Roundabout at ramp terminal	3.5
Roundabout at surface road	3.2

3.2.3 Vehicle-to-Vehicle Yielding Behavior

3.2.3.1 Aggregated Data Summary

The recorded videos on two approaches at each of the sites were reviewed to obtain data related to driver yielding behavior towards vehicles in the traffic circle. As stated earlier, vehicles approaching the roundabout under free-flow conditions and attempting to enter the roundabout when there is a vehicle approaching from the left in the traffic circle were identified. The yield response of such vehicles was recorded as a binary response (1 if yielded, 0 if not) along with other behavioral data as alluded to previously. Table 11 shows the aggregated results of yielding behavior at each of the sites included in the study. Sites marked with an asterisk are the sites where one of the approaches investigated for yielding behavior is an exit ramp from a freeway facility. Most of the sites showed yielding rates exceeding 95%. Sites 4, 11, 12, and 13 showed non-yielding rates exceeding 10%. Two of these sites were on ramp terminals. It should be noted that the posted speed limit on the approaches where the data were collected was 40 mph or higher on all the four sites where non-yielding rates exceeded 10%.

Table 11 Vehicle-to-Vehicle Yielding Behavior at Roundabouts by Site

Site ID	Site	Sample Size	Yielded	Did not Yield
1	M-5 at Pontiac Trail	190	97.4%	2.6%
2a	NB I-75 at Monroe M-46*	77	100%	0%
2b	SB I-75 at Monroe M-46	36	94.4%	5.6%
3a	NB I-75 at M-81*	168	100%	0%
3b	SB I-75 at M-81*	204	98.0%	2.0%
4	US-10 at M-30*	129	80.6%	19.4%
5	US-127 BR at Mission Road	24	91.7%	8.3%
6	US-23 at Lee Road*	177	100%	0%
7	EB I-94 at Sprinkle Road*	191	95.8%	4.2%
8	WB I-94 at Sprinkle Road	100	100%	0%
9	US-10 BR/M-20 at Patrick Road	53	90.6%	9.4%
10	NB I-75 at Bristol Road	159	94.3%	5.7%
11	US-23 at Geddes Road*	199	82.4%	17.6%
12	M-52 at Werkner Road	87	87.4%	12.6%
13	Farmington at Maple Road	200	89.0%	11.0%
14	Drake at Maple Road	98	95.9%	4.1%
15	Geddes at Earhart Road	129	99.2%	0.8%
16	M-53 at 26 Mile Road*	156	98.1%	1.9%

*One of the approaches is an exit ramp

The yielding behavior was also investigated based on site characteristics. The yielding behavior of vehicles were grouped based on the type of approach- major road, minor road, or exit ramp. Table 12 shows the yield rates based on approach type. Vehicles on minor approaches showed higher yielding rates compared to the other two categories which could primarily be driven by higher traffic volumes on major road and higher speeds of vehicles entering the roundabout from the exit ramp. Another interesting trend for yield behavior was observed based on the lane position of vehicles as shown in Table 13. When the number of approach lanes were 2 or more, vehicles in the rightmost lanes tended to yield less frequently compared to middle and left lane. Also, multilane roundabouts exhibited higher yielding rates compared to single lane roundabouts, as shown in Table 14. Drivers may slow down, in part, because multilane roundabouts tend to be more complex to navigate. This includes determining the appropriate lane that corresponds to their intended movement. Such differences may help to explain these higher yielding rates. Finally, sites with pedestrian crossing facilities showed lower yield rates compared to sites without any such provisions as presented in Table 15. Sites with pedestrian crossings generally tended to be in urban areas with higher volume and lower speeds, which tends to explain the lower yielding rates at these sites. However, when comparing yield rates based on roadway context (rural vs urban), the yield rates for roundabouts in urban areas were slightly higher than in rural areas.

Table 12 Vehicle-to-Vehicle Yielding Rates based on Approach Type

Approach Type	Sample Size	Yielded	Did not Yield
Major road	1,397	93.1%	6.9%
Minor road	359	97.8%	2.2%
Exit ramp	621	94.8%	5.2%

Table 13 Vehicle-to-Vehicle Yielding Rates based on Lane Position

Number of Approach Lanes	Lane Position	Sample Size	Yielded	Did not Yield
One	Right	852	93.3%	6.7%
Two	Left	638	96.6%	3.4%
Two	Right	389	92.5%	7.5%
Three	Left	308	93.8%	6.2%
Three	Middle	155	96.1%	3.9%
Three	Right	35	91.4%	8.6%

Table 14 Vehicle-to-Vehicle Yielding Rates based on Number of Circulating Lanes

Number of Circulating Lanes	Sample Size	Yielded	Did not Yield
One	1,093	92.4%	7.6%
Two	1,094	95.6%	4.4%
Three	190	97.4%	2.6%

Table 15 Vehicle-to-Vehicle Yielding Rates based on Presence of Pedestrian Crossing

Pedestrian Crossing	Sample Size	Yielded	Did not Yield
Present	675	90.8%	9.2%

Pedestrian Crossing	Sample Size	Yielded	Did not Yield
Not present	1,702	95.7%	4.3%

3.2.3.2 Statistical Analysis

The yield rate summary provided in the previous section only accounts for one site characteristic at a time. To investigate the likelihood of driver yielding to another vehicle in the traffic circle, while controlling for other site-specific characteristics, statistical models were developed. To that end, a logistic regression model for vehicle-to-vehicle yielding was developed. For each of the vehicles observed, the response variable was coded as a binary variable to indicate whether the vehicle yielded to traffic in circle or not. The binary logistic model takes the form as shown in Equation 9.

$$Y_i = logit(P_i) = ln\left(\frac{P_i}{1-P_i}\right) = \beta_o + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_k X_k \qquad \text{Equation 9}$$

Where,

P_i = probability of i^{th} vehicle yielding to traffic,

X_1 to X_k = a series of predictor variables that are associated with the yielding behavior (e.g., traffic volume, site characteristics, etc.), and

β_1 to β_k = a series of estimable parameters.

Since multiple observations are recorded at each of the sites, there may be some correlation among observations due to unobserved site-specific characteristics. Thus, random effects modeling framework is adopted to account for these correlations. Random effects are considered by re-writing the constant term in Equation 9 as follows:

$$\beta_{0i} = X\beta_0 + \omega_i \qquad \text{Equation 10}$$

Where,

ω_i = randomly distributed random effect for site.

Table 16 presents the descriptive statistics of pertinent variables investigated in the statistical analysis. The descriptive statistics show that the overall yielding rate across all sites was 94%. Nearly half of the observations were recorded in urban context (mean = 0.46), and 93% of the sites had warning sign present.

Table 16 Descriptive Statistics of Pertinent Variables for Vehicle-to-Vehicle Yielding Dataset

Parameter	Min	Max	Mean	Std. Dev.
Yield (1 if yes, 0 otherwise)	0	1	0.94	0.23

Parameter	Min	Max	Mean	Std. Dev.
Approach Class				
Major (1 if yes, 0 otherwise)	0	1	0.59	0.49
Minor (1 if yes, 0 otherwise)	0	1	0.15	0.36
Ramp (1 if yes, 0 otherwise)	0	1	0.26	0.44
Roundabout Diameter (ft)				
<=100 (1 if yes, 0 otherwise)	0	1	0.42	0.49
>100 and <=150 (1 if yes, 0 otherwise)	0	1	0.25	0.43
>150 (1 if yes, 0 otherwise)	0	1	0.33	0.47
Other Site Characteristics				
Approach AADT	1,781	17,381	8,558.39	4,071.07
Urban context (1 if yes, 0 otherwise)	0	1	0.46	0.50
Warning sign present (1 if yes, 0 otherwise)	0	1	0.93	0.25
Exclusive right turn lane (1 if yes, 0 otherwise)	0	1	0.45	0.50
Sample size	2,358			

As the maximum diameter of the inscribed circle of roundabout exceeded 100 ft, the likelihood of drivers yielding to traffic in circle reduced, although these results were not statistically significant. If the drivers were informed of the presence of the roundabout ahead of time, their likelihood of yielding increased significantly. Also, roundabout sites with an exclusive right turn lane showed higher yield rates compared to sites which had no such provisions.

The results showed a very strong relationship between the time to approaching vehicle from the left and yielding behavior. Yielding consistently increased as the time gap between subject vehicle and conflicting vehicle approaching from the left decreased. This is reflective of fundamental driver behavior, including both risk-taking behavior and driving experience. Several other variables were also examined, including approach speed limit, approach-specific traffic volumes, and number of approach lanes. However, none of these relationships were found to be statistically significant.

Table *17* presents the results of the random effects logistic regression model for yielding behavior of vehicles towards other vehicles in the traffic circle. The table shows parameter estimates along with standard errors and the p-value. A p-value of less than 0.05 indicates that the parameter estimate is statistically significant at 95% confidence interval. When interpreting the model, a positive parameter estimate indicates that the likelihood of the driver yielding increases as that variable increases, while the opposite is true for a negative parameter estimate. To assist in the interpretation, odds ratio (OR) are also provided which represent the change in odds of yielding due to the parameter.

The results show that the drivers entering the roundabout through a minor approach or a ramp were more likely to yield to traffic inside the circle compared to vehicles entering the roundabout through a major approach. This is reflective of the volume on these approaches. As discussed previously, higher traffic volume on the major approach (by definition) forces the vehicles on the minor approach to yield to traffic coming from the major approach. Roundabouts in urban areas exhibited higher yielding rates compared to the ones in rural areas. This trend may be due to higher traffic volumes in urban areas compared to rural areas and also due to fewer gaps available at urban roundabout. As the maximum diameter of the inscribed circle of roundabout exceeded 100 ft, the likelihood of drivers yielding to traffic in circle reduced, although these results were not statistically significant. If the drivers were informed of the presence of the roundabout ahead of time, their likelihood of yielding increased significantly. Also, roundabout sites with an exclusive right turn lane showed higher yield rates compared to sites which had no such provisions.

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Table 17 Random Effect Logistic Regression Model Parameter Estimates for Vehicle-to-Vehicle Yielding

Parameter	Estimate	Standard Error	p-value	OR
Intercept	-2.539	1.937	0.190	n/a
Approach Class				
Major		*Baseline*		
Minor	2.470	1.416	0.081	11.822
Ramp	1.249	0.671	0.063	3.487
Inscribed Diameter (ft)				
<=100		*Baseline*		
>100 and <=150	-1.302	0.740	0.078	0.272

Parameter	Estimate	Standard Error	p-value	OR
>150	-1.483	0.802	0.065	0.227
Context				
Rural		*Baseline*		
Urban	1.418	0.759	0.062	4.129
Roundabout Warning Sign				
Not present		*Baseline*		
Present	2.035	1.740	0.242	7.652
Exclusive Right Turn at Roundabout				
No		*Baseline*		
Yes	1.245	0.557	0.025	3.473
Time to Vehicle from Left (s)				
>6		*Baseline*		
5-6	2.387	0.649	<0.001	10.880
4-5	1.102	0.476	<0.001	3.010
3-4	2.849	0.464	<0.001	17.270
2-3	3.255	0.432	<0.001	25.920
1-2	3.378	0.405	0.0207	29.312
<=1	4.813	0.407	<0.001	123.100
Random Effect: Variance of intercept	1.306			

3.2.4 Vehicle-to-Pedestrian Yielding Behavior

Similar to the case of vehicle-to-vehicle yielding behavior, the vehicle-to-pedestrian yielding behavior was investigated. The following sub-sections present the aggregated data summary and the results from the statistical analysis that investigated what factors increase the likelihood of drivers yielding to pedestrians at roundabout crossing.

3.2.4.1 Aggregated Data Summary

The data related to yielding towards pedestrians were collected on five sites where pedestrian crossing facilities were provided on at least one of the roundabout approaches. As stated previously, at each of these sites, 50 pedestrian crossing events were staged and data were collected with respect to whether the driver yields to pedestrian or not. During each pedestrian crossing event, multiple vehicles can interact with the pedestrian, especially if the approach has 2 or more lanes. While extracting the data, each vehicle to pedestrian interaction was recorded separately. Thus, a single crossing event generated one or more records in the data, thereby increasing the sample size to more than 50.

Table 18 shows the yield rates on each of the five sites. Two of the sites were on exit ramps and exhibited much lower yield rates towards pedestrians compared to other sites. This is expected due to higher speeds of vehicles exiting the freeway and thus failing to yield to pedestrian standing at the curb.

Table 18 Vehicle-to-Pedestrian Yielding Behavior at Roundabouts by Site

Site ID	Site	Sample Size	Yielded	Did not Yield
11	US-23 at Geddes Road*	58	44.8%	55.2%
13	Farmington at Maple Road	93	89.2%	10.8%
14	Drake at Maple Road	81	90.1%	9.9%
15	Geddes at Earhart Road	81	86.4%	13.6%
16	M-53 at 26 Mile Road*	37	43.2%	56.8%

*Exit ramp

Two of the sites- Farmington at Maple Rd, and Drake at Maple Rd had PHBs installed and thus exhibited higher yield rates (nearly 90%) towards pedestrians. The sites were also grouped based on their context (urban vs rural). Urban roundabouts exhibited higher yield rates towards pedestrians (90%) compared to rural roundabouts (64%). Both contexts had similar sample size of vehicle to pedestrian interactions.

Additional investigation into driver behavior at two sites with PHB was carried out. A PHB signal has three main phases- yellow phases (flashing followed by solid), solid red, and flashing red. The yellow phase indicates the motorists that the signal has been activated and they should prepare to stop. During solid red phases, the motorists are required to come to a complete stop at the stop line and yield to pedestrians. During flashing red phases, the motorists are required to stop and yield to pedestrians in the crosswalk, and can proceed to enter the roundabout if no pedestrian is using the crosswalk. Table 19 shows the driver response to three phases of PHB signal combined for both the sites with PHB installed. The sample size corresponds to the number of vehicles that arrived at the PHB signal during that phase. The behavior of only the lead vehicle approaching the roundabout was recorded. Drivers are not legally required to yield or stop during the yellow phase of the PHB signal. However, the results from Table 19 show that 41% of the drivers came to a full stop during yellow phase. Moreover, 9% of the motorists simply proceeded through during solid red phase when they were required to legally come to a full stop. This indicates motorists may not be familiar with the PHB signal and there may be a need to educate drivers regarding how to navigate through intersections installed with such facilities.

Table 19 Driver Behavior During Different Phases of PHB

PHB Phase	Sample Size	Proceed	Slow Down	Stop Before or At Stop Line	Stop After Stop Line
Yellow	122	52.5%	6.6%	18.9%	22.1%
Solid red	155	9.0%	6.5%	34.2%	50.3%
Flashing red	157	77.7%	2.5%	10.2%	9.6%

3.2.4.2 Statistical Analysis

To investigate the driver's likelihood to yield to a pedestrian at roundabout crossings, statistical models were developed. Table 20 presents the descriptive statistics of the pertinent variables used in the analysis. A similar methodology was adopted as in the case of vehicle-to-vehicle yielding as discussed in Section 3.2.3.2. However, the data did not show enough variation in terms of site-specific characteristics to estimate a random effects model. Hence, simple logistic regression was estimated. The results of the regression model are presented in Table 21. The dependent variable is a binary indicator that indicates whether the motorist yielded to the pedestrian intending to cross the roundabout crossing. The interpretation of the parameter estimates is same as explained previously in Section 3.2.3.2

Table 20 Descriptive Statistics of Pertinent Variables for Vehicle-to-Pedestrian Yielding Dataset

Parameter	Min	Max	Mean	Std. Dev.
Yield (1 if yes, 0 otherwise)	0	1	0.77	0.42
Site-Specific Characteristics				
Has PHB (1 if yes, 0 otherwise)	0	1	0.50	0.50
On exit ramp (1 if yes, 0 otherwise)	0	1	0.27	0.45
Traditional (1 if yes, 0 otherwise)	0	1	0.23	0.42
Type of Pedestrian Warning Sign				
In-street sign (1 if yes, 0 otherwise)	0	1	0.73	0.45
Yield here to ped (1 if yes, 0 otherwise)	0	1	0.17	0.37
Traditional yellow diamond (1 if yes, 0 otherwise)	0	1	0.11	0.31
Other Site Characteristics				

Parameter	Min	Max	Mean	Std. Dev.
Approach AADT	3,128	11,161	8,162.64	2,911.83
Urban context (1 if yes, 0 otherwise)	0	1	0.50	0.50
Roundabout diameter > 150 ft (1 if yes, 0 otherwise)	0	1	0.50	0.50
Multilane roundabout (1 if yes, 0 otherwise)	0	1	0.60	0.49
Vehicle Characteristics				
Heavy vehicle (1 if yes, 0 otherwise)	0	1	0.03	0.17
Vehicle in near lane (1 if yes, 0 otherwise)	0	1	0.56	0.50
Sample size	350			

Table 21 Simple Logistic Regression Model Parameter Estimates for Vehicle-to-Pedestrian Yielding

Parameter	Estimate	Standard Error	p-value	OR
Intercept	1.112	0.477	<0.001	n/a
Site-Specific Characteristics				
Conventional		*Baseline*		
Has PHB	0.868	0.490	0.076	2.38
On exit ramp	-1.785	0.415	<0.001	0.17
Vehicle Type				
Passenger car		*Baseline*		
Heavy vehicle	-0.937	0.771	0.224	0.39
Vehicle Position				
Far lane		*Baseline*		
Near lane	0.786	0.347	0.024	2.19

The model results show that drivers were more likely to yield to pedestrians at sites with PHB which is expected since placement of exclusive pedestrian signal increases their compliance to yield. Vehicles exiting a freeway were 83% less likely to yield to pedestrians at roundabouts located on exit ramps compared to a traditional roundabout located at crossing of two arterials. This trend may be explained based on the speed selection behavior modeled previously which showed higher speeds on sites on exit ramps. Also, as expected, heavy vehicles were 61% less likely to yield to pedestrians at roundabout crossings compared to passenger cars. And lastly, vehicles in the lane near to the pedestrian were 119% more likely to yield to pedestrians compared to vehicles in the far lane.

Several other site characteristics such as approach speed limit, type of pedestrian sign, etc., were correlated with other variables due to low number of sites and thus were not included in the model.

3.3 Summary

To investigate driver behavior at roundabout entry, a list sites for field data collection was prepared. Extensive field data collection related to driver speed profile when approaching a roundabout, their gap acceptance and rejection behavior, driver yielding behavior towards other vehicles in roundabout circle, and yielding behavior towards pedestrians at roundabout crossing were collected. Results showed that driver speed selection largely depended upon vehicle type, traffic volume on the cross road, and its distance from yield line. In terms of gap behavior, drivers tend to accept larger gaps on single lane roundabouts compared to multilane roundabouts. Similarly, roundabouts in urban areas tended to show larger critical gaps compared to roundabouts in rural areas.

When yielding to traffic in circle, drivers were more likely to yield if the roundabout is in an urban area, when a roundabout approach has a warning sign, or when the diameter of the roundabout is lesser. Also, as the time to the vehicle approaching from the left decreases, the likelihood to yield increases. Drivers entering the roundabout through a minor approach or an exit ramp were more likely to yield to traffic compared to drivers entering through a major road.

When yielding to pedestrians at roundabout crossings, drivers tended to yield more on sites that had dedicated pedestrian signals such as a PHB. Roundabouts on interchanges exhibited much lower yield rates compared to roundabouts not on interchange. Also, heavy vehicles were less likely to yield to pedestrian compared to passenger car. Lastly, vehicles in lane near to the pedestrian were more likely to yield.

4 SAFETY PERFORMANCE OF ROUNDABOUTS

Literature has shown roundabouts to improve safety performance and reduce the severity of crashes. To that end, comprehensive safety analysis of Michigan roundabouts was conducted. The following sections detail the data collection, integration and analysis methodology followed by analysis results of safety performance of roundabouts. The results are also compared with the prior MDOT study on roundabout safety (Bagdade et al., 2011).

4.1 Data Collection and Preparation

4.1.1 Site Selection

For the purposes of safety analysis, a comprehensive list of all known roundabouts in Michigan was prepared, with an initial list of roundabouts obtained from WSP. This database was then supplemented by additional data sources including MDOT, the national roundabout database maintained by Kittelson & Associates (Kittelson & Associates, Inc., n.d.), and Michigan roundabout database maintained by Michigan Auto Law ("Michigan Roundabouts Resource Center," n.d.). A total of 180 roundabouts were identified in this manner. The dataset included the name of the intersecting roads, GIS coordinates of the roundabout, the year in which the roundabout was constructed, and some geometric characteristics including number of legs and number of circulating lanes.

For each of the roundabouts, a companion control site was also identified which was an intersection in the vicinity of the roundabout. The control sites were selected in such a manner that the traffic volume and geometric characteristics such as number of legs were same as that of the roundabout. Also, the type of control at the control site (stop control or signalized) was also same as that at the roundabout location prior to the construction of the roundabout. However, there were a few cases where it was not feasible to identify a suitable control site for the roundabout. For example, roundabouts with 5 or 6 legs, or multiple roundabouts in close vicinity on exit ramps or dense residential areas, etc. Figure 16 shows examples of roundabouts where it was not feasible to identify suitable control sites. The image on the left shows a roundabout with 6 approach legs for which no suitable control site was identified. The image on the right shows 3 roundabouts in close vicinity out of which a control site was included for only one of the roundabouts. Thus, the final dataset included 157 control sites as compared to 180 roundabout locations.

Figure 16 Example of Roundabouts with No Suitable Control Site

4.1.2 Collection of Site Geometry Data

Site geometry characteristics tend to influence safety at intersections, roundabouts included. Figure 17 shows the typical geometry of a modern roundabout (Robinson et al., 2000). For the purposes of safety analysis of roundabouts, roadway geometry data were collected for all the roundabouts as well as the control sites using Google Maps aerial imagery and street view. Some of the data were also available through the dataset provided by WSP and the RAP. Following geometric data were collected as a part of the data collection process:

- For roundabouts:
    - Location characteristics: coordinates, county, and names of intersecting roads
    - Type of control prior to roundabout construction
    - Number of approach legs
    - Number of circulating lanes
    - Minimum and maximum widths of circulating lane
    - Diameter of inscribed circle
    - Diameter of central island

- Number of approach lanes and their widths separately on major approach and minor approach
- Presence of exclusive right lane separately on major approach and minor approach (separated by physical island)
- Minimum and maximum entry width and exit width
- Whether the roundabout is located on an interchange
- Roadway context (urban or rural)
- Speed limit separately on major approach and minor approach
- Presence and type of pedestrian control (none, signage, PHB)
- Whether the roundabout provides direct access to a parking lot or a small residential area
- Number of access points (driveways) within 250 ft from yield line separately on major approach and minor approach

- For control sites:
    - Location characteristics: coordinates, county, and names of intersecting roads
    - Type of traffic control (stop controlled or signalized)
    - Number of approach legs
    - Number of approach lanes and their widths separately on major approach and minor approach
    - Number of exclusive right and exclusive left turn lanes separately on major approach and minor approach
    - Whether the intersection is located on an interchange
    - Roadway context (urban or rural)
    - Speed limit separately on major approach and minor approach
    - Presence and type of pedestrian control (none, signage, signal phase)
    - Number of access points (driveways) within 250 ft of stop line separately on major approach and minor approach

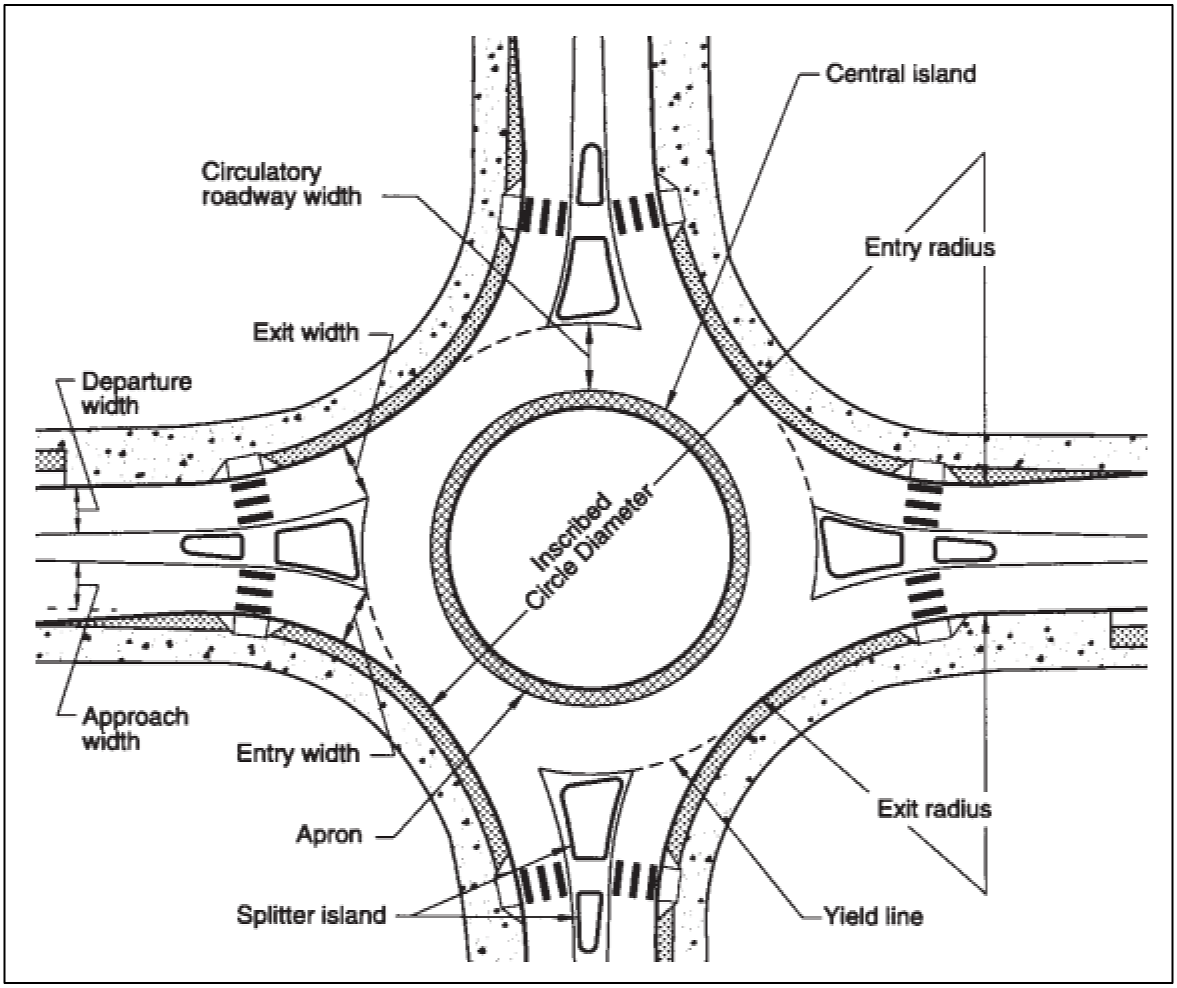

Figure 17 Basic Geometric Elements of a Modern Roundabout *(Robinson et al., 2000)*

4.1.3 Collection of Traffic Volume Data

The traffic volume entering the roundabout is an important factor to consider during safety analysis. Thus, the annual average daily traffic (AADT) on each of the approaches of the roundabout and corresponding control sites was obtained manually using the Transportation Data Management System (TDMS) maintained by MS2 (*Transportation Data Management System*, 2022), with access to the data obtained through MDOT. The portal provides traffic counts from count stations located throughout Michigan. Figure 18 shows a snapshot of user interface of the MDOT TDMS traffic count portal. The figure shows the roundabout at the intersection of Maple Road and Drake Road (indicated by green arrow). Count stations are located in the vicinity of the roundabout on each of the roads. The count station north of the roundabout on Drake road shows two-way AADT counts starting 2018 to 2021. The AADT values for 2018 and 2019 were recorded in the dataset. Similarly, the data from count station on the other approaches was also extracted.

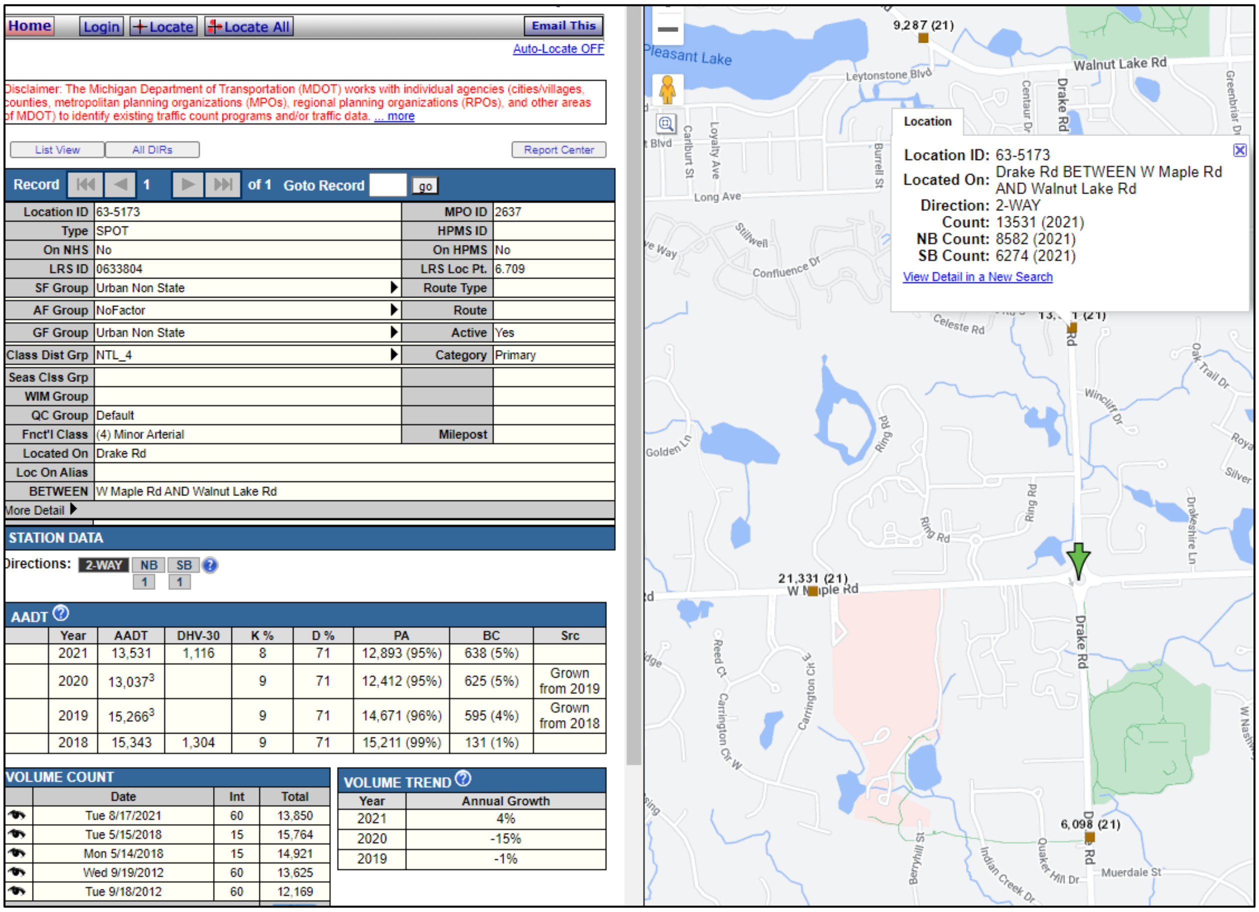

Figure 18 User Interface of MDOT TDMS Traffic Count Portal

There were many sites where count stations were not available in the vicinity of the intersection on the MDOT TDMS portal. As such, alternate sources of AADT data were explored. Several cities and counties in the state of Michigan also maintain a similar TDMS count data portal MS2. Some of the missing AADT data were obtained from these public portals. The list of these cities and county road commissions is as follows:

- City of Lansing
- Grand Valley Metropolitan Council (GVMC)
- Kalamazoo Count Road Commission
- Livingston Count Road Commission
- Macomb Count Road Commission

- Oakland Count Road Commission
- Southeast Michigan Council of Government (SEMCOG)
- Tri-County Regional Planning Commission
- Washtenaw Count Road Commission
- West Michigan Shoreline Regional

For each of the approaches of the roundabouts and the control sites included in the analysis, the two-way AADT was obtained manually for the analysis period from 2004 to 2019. In most of the cases, the AADT information was not available for all years. In cases where the AADT information was missing for intermediate years, the AADT was linearly interpolated. For example, if AADT is available for 2014 and 2017, but not for the years in between, then the AADT for 2015 and 2016 was linearly interpolated. In other cases where AADT was not available for prior years, a 1 percent growth rate was assumed to extrapolate the AADT data. Even after obtaining AADT data from TDMS portals maintained by various transportation agencies, the AADT information was not available for some of the sites, particularly in highly rural areas, or sites located on minor streets.

4.1.4 Collection of Crash Data

A total of 16 years of crash data (2004-2019) were obtained from Michigan State Police (MSP). The year of 2020 data was excluded due to the impacts of the COVID-19 pandemic. The MSP database included details of crash-, vehicle-, and person-level information corresponding to each police-reported crash that occurred in Michigan over this time period. Information such as the worst level of injury sustained in the crash based on the KABCO scale (K-fatal injury, A-incapacitating injury, B-non-incapacitating evident injury, C-possible injury, O-no injury) and the time-of-day when the crash occurred were among the primary factors of interest.

The crashes occurring on the roundabouts and the control intersection sites were identified using ArcGIS Pro. Specifically, polygons were manually drawn based on the layout of each roundabout and reference sites using a 300 ft. radius buffer to locate the crashes. Several concerns were addressed while drawing these polygons to avoid over or under-estimating the crashes. First, the cases where sites were located close another major intersection were identified. The polygons for such sites were cut before reaching adjacent intersections even if the 300 ft. radius was not met. Figure 19 shows an example of such case where the 300 ft buffer was not met on one of the three legs of roundabout.

Figure 19 Example of Roundabout Adjacent to Another Intersection

Second, if two study sites were closely spaced or located side by side and shared the same leg, the polygons for each location at that specific leg were stopped halfway to ensure the polygons do not overlap. An example of such a case is shown in Figure 20. Lastly, the geometric layouts of several target sites changed after constructing the roundabouts. For these sites, different polygons were plotted for the before and after periods as shown in Figure 21.

After drawing the polygons for each of the roundabout as well as the control site intersections, crash data were merged spatially by crash severity separately for each year. However, during initial quality checks of the merged crash data it was found that the sites located on interchange were also capturing the crashes that occurred on the freeway mainline since the crashes cannot be separated into whether they occurred on the freeway mainline or the crossroad based on the GIS coordinates alone. Thus, for all the crashes that were captured in the polygons of sites on interchanges, crash reports were reviewed to eliminate the crashes that actually occurred on the freeway mainline.

For roundabouts that have different number of legs through which traffic can enter the roundabout compared to the total number of approach legs, (for example, roundabout shown in Figure 3), the crashes were counted on all approach legs. That means, in the analysis, total number of legs were considered instead of total number of legs through which traffic can enter the roundabout.

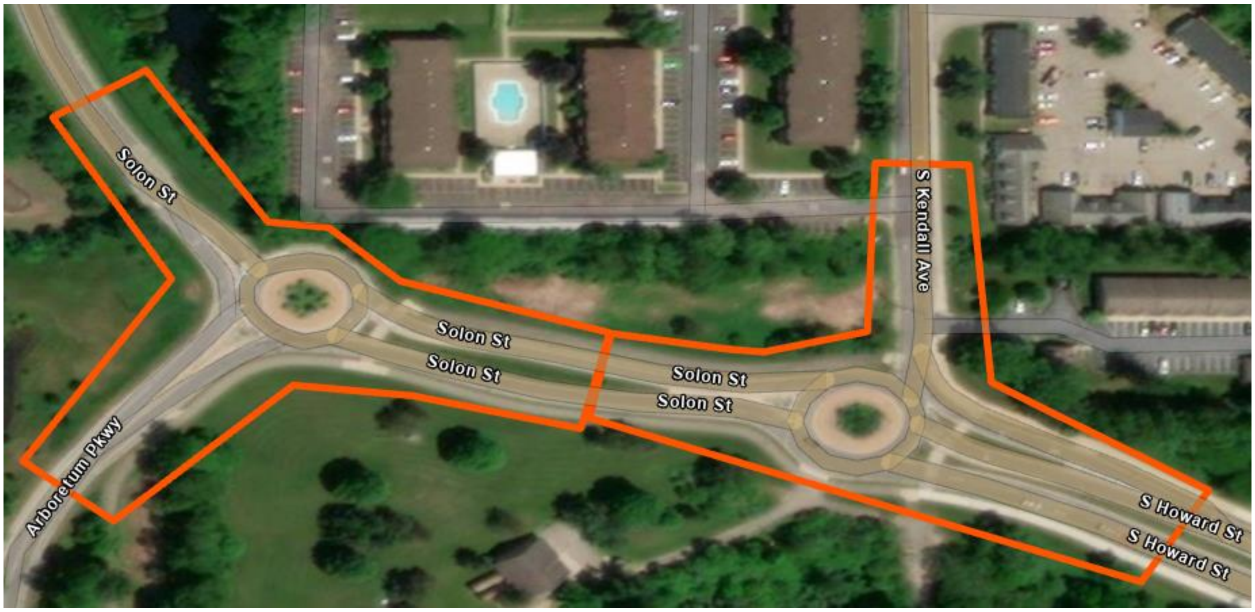

Figure 20 An Example of Two Adjacent Roundabouts

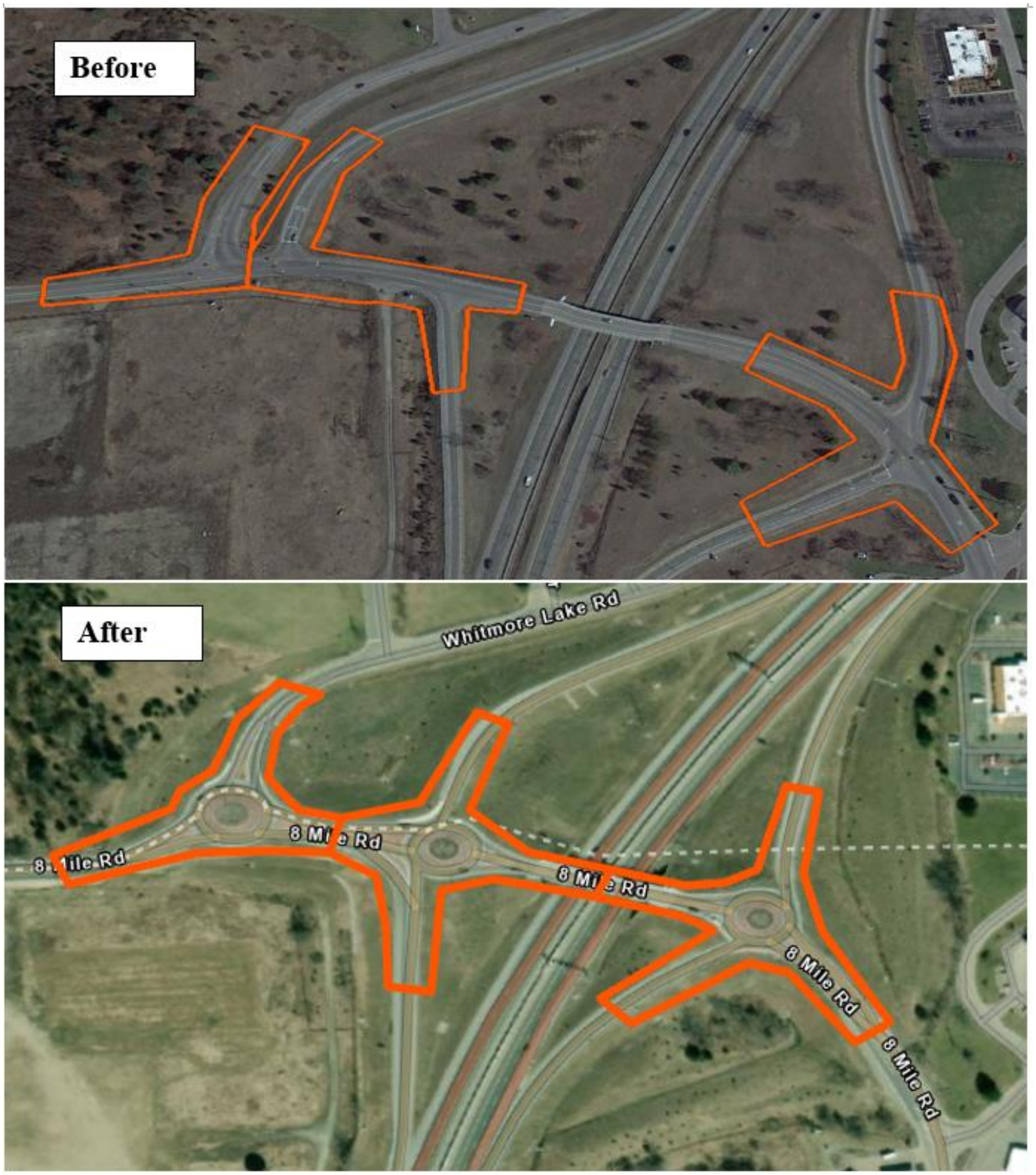

Figure 21 An Example of Roundabout with Change in Geometric Layout

4.1.5 Data Integration and Preparation

As stated earlier, the analysis period was from 2004-2019. For each of the roundabout and control sites, the geometric data was merged first since it remained constant across all years. This was followed by integrating traffic volume data which included AADT on all approaches separately. Since the AADT collected was 2-way traffic data, the total entering volume (TEV) was calculated by adding the AADT on all the approaches and dividing by 2. These data were merged separately for each of the study period. Similarly, number of crashes that occurred on each of the sites was merged by severity level for each of the study period. This resulted in an intersection level dataset with one row representing one site-year. This was the base dataset for analysis purposes. Separate datasets were prepared depending upon the safety analysis by applying appropriate filters to the base dataset as discussed later in the respective safety analysis sections.

4.2 Aggregate Safety Data Summary

The initial step in safety data analyses was to investigate crash data at aggregate level. This included computing annual average crash frequency for roundabouts by site, severity, and crash type. To reiterate, the analysis period of this study was from 2004-2019. The following sub-sections detail the process of comparing crash frequencies at various levels of detail and present the results.

4.2.1 Comparison of Annual Average Crash Frequencies

The average crashes for each roundabout before and after construction were computed. The average number of crashes before the conversion was calculated by adding all the crashes from 2004 to the year of construction and dividing the total by the number of years. Similarly, the average number of crashes after construction was computed by adding up all crashes from the year the roundabout became operational to 2019. The crashes that occurred in the year of constructing the roundabout were excluded from the analysis because the exact date of roundabout became operational was unknown and including the construction year in the analysis could bias the results. This analysis was done for the total crashes, injury crashes (ABC), and fatal and A-level crashes (KA). The frequencies were compared on each of the 117 roundabouts with single circulating lanes, 49 double circulating lanes, and 8 triple circulating lanes.

It should be noted that 16 single-lane and 2 double-lane roundabouts do not have any before-construction crash data because they were constructed before our analysis period of 2004 – 2019. Also, 9 single and 4 double lanes roundabouts do not have after-conversion crash data because they were constructed after 2019. Similarly, the construction year was unknown for 6 roundabouts, and crashes could not be obtained for before and after periods. These sites were excluded from our analysis.

The results showed that 48 out of 92 (52%) sites with single circulating lanes showed an increase in total average crashes. For comparison purposes, the prior MDOT study published in 2011 (Bagdade et al., 2011) showed an increase in total crashes on only 19% of their sites (4 roundabouts out of 21 total sites). Similarly, the present study showed that injury crashes increased on 30 roundabout sites (33%) after their construction, compared to only 4 sites (19%) in the MDOT 2011 study that exhibited an increase in injury crashes. For the average fatal and A-level crashes, 16 (17%) roundabouts showed a slight increase in the present study. In contrast, no increase was recorded in the MDOT study compared to before and after the construction period.

For roundabouts with double circulating lanes, 33 out of 42 (79%) roundabouts showed an increase in total crashes in the present study. Similar results were obtained in the 2011 MDOT study that showed an increase on 9 out of 13 (69%) sites after construction. A similar trend was observed in the injury crashes, where 13 (31%) roundabouts showed an increase after construction compared to 3 (23%) in the 2011 MDOT study. In the case of fatal and A-level crashes, an increase was observed on 16 (39%) sites in the current study compared to 3 (23%) in the 2011 MDOT study.

For triple-lane roundabouts, an increase in total crashes was observed on all 8 roundabouts in this study. In comparison, the 2011 MDOT study showed that the total crashes increased on only 6 out of 8 triple lane roundabouts. However, 4 (50%) sites showed an increase in average injury crashes in this study as compared to only 1 (2%) in the MDOT 2011 study. Additionally, 25% and 50% of roundabouts sites showed an increase in fatal and A-level crashes after construction in the present study and the 2011 MDOT study, respectively.

These differences in trends observed between the present study and the MDOT 2011 study arise from the difference in the analysis period, differences in number of roundabouts, and the increase in AADT over the years. The effect of increasing traffic volume was not accounted for in this analysis. For site-by-site comparisons, before and after annual average crash frequencies by site are presented in Appendix-B.

Table 22 presents summary of total average crashes by roundabout type. These statistics were computed by adding up all the crashes and dividing by the number of years. All crash metrics presented for single lane showed a decrease. For double lanes, while total crashes increased because of an increase in PDO crashes, injury crashes and fatal and A-level crashes saw a reduction. Triple lane roundabouts saw an increase in total and injury crashes with a reduction in only fatal and A-level crashes. Overall, the average of all roundabouts showed that total crashes increased by 58%, injury crashes reduced by 72%, with fatal and A-level crashes reduced by 50%.

Table 22 Annual Average Crash Frequency by Roundabout Type

Roundabout Type	Number of Sites	Total Crashes		Injury Crashes		KA Crashes	
		Before	After	Before	After	Before	After
Single lane	92	4.10	3.16	0.86	0.36	0.09	0.04
Double lane	42	10.91	18.49	2.26	1.50	0.17	0.09
Triple Lane	8	26.54	67.14	5.17	5.18	0.25	0.11
All Roundabouts	142	6.65	11.55	1.38	0.99	0.12	0.06

4.2.2 Crash Severity Trends

One of the primary safety benefits of roundabouts is that they tend to reduce crash severities due to lower speeds at which drivers are forced to enter the intersection and traffic flowing around a central circle in only one direction. As such, the crash severity distribution before and after roundabout construction was also investigated. In describing the injury severity levels, single-lane and double-lane roundabouts were combined. Table 23 shows that fatal crashes accounted for 0.26% and 0.06% of total crashes before and after the construction of roundabouts, respectively. Injury crashes showed the most decrease, with 20.86% before construction and 9.20% after construction. As expected, the proportion of PDO crashes showed the most increase, before and after construction. These results and trends are similar to the MDOT 2011 study.

For the 8 triple lane roundabouts available in this study, the results showed that there was no change in fatal injury crash trends before and after construction. However, the proportion of injury crashes decreased from 19.47% before construction to 7.71% after construction. As expected, there was an 11.75% increase in the proportion of PDO crashes after the construction of the roundabouts. These trends are also similar to the MDOT 2011 study.

Table 23 Crash Severity Distribution Before and After Roundabout Construction

Severity Type	Before Period		After Period	
	Frequency	Percentage of Total	Frequency	Percentage of Total
Single and Double Lane Roundabouts				
Fatal	21	0.26%	5	0.06%
A-Level	129	1.62%	58	0.65%
B-Level	374	4.71%	192	2.14%
C-Level	1154	14.53%	577	6.42%
PDO	6265	78.87%	8160	90.75%
Total	7943	100.00%	8992	100.00%
Triple Lane Roundabouts				
Fatal	0	0.00%	0	0.00%
A-Level	6	0.94%	11	0.17%
B-Level	18	2.83%	77	1.19%
C-Level	100	15.70%	409	6.35%
PDO	513	80.53%	5948	92.29%
Total	637	100.00%	6445	100.00%

4.2.3 Differences by Type of Crash

Conversion of conventional intersections to a roundabout also helps in reducing certain types of intersection crashes such as head-on crashes. Figure 22 shows the changes in the proportion of various roundabout crash types before and after the construction of roundabout. The trends are shown separately for single lane, double lane, and triple lane roundabouts.

The results showed that the construction of a roundabout significantly reduced the number of angle crashes and head-on crashes on single lane roundabouts. Angle crashes were reduced by the greatest margin (8.5%). On double and triple lane roundabouts, however, the proportion of angle crashes increased. The proportion of sideswipe crashes increased across all categories which is expected since this type of crash is more common on roundabouts. Single vehicle crashes, which involve crashes with fixed objects, also increased after the roundabout construction. On single lane roundabouts, the proportion of single vehicle crashes increased by 11%. Lastly, rear-end crashes, which includes rear-end crashes on left or right turn reduced significantly across all categories of roundabout. These trends are largely similar to the 2011 MDOT study which showed reduction in rear end, angle, head-on crashes, and an increase in sideswipe crashes.

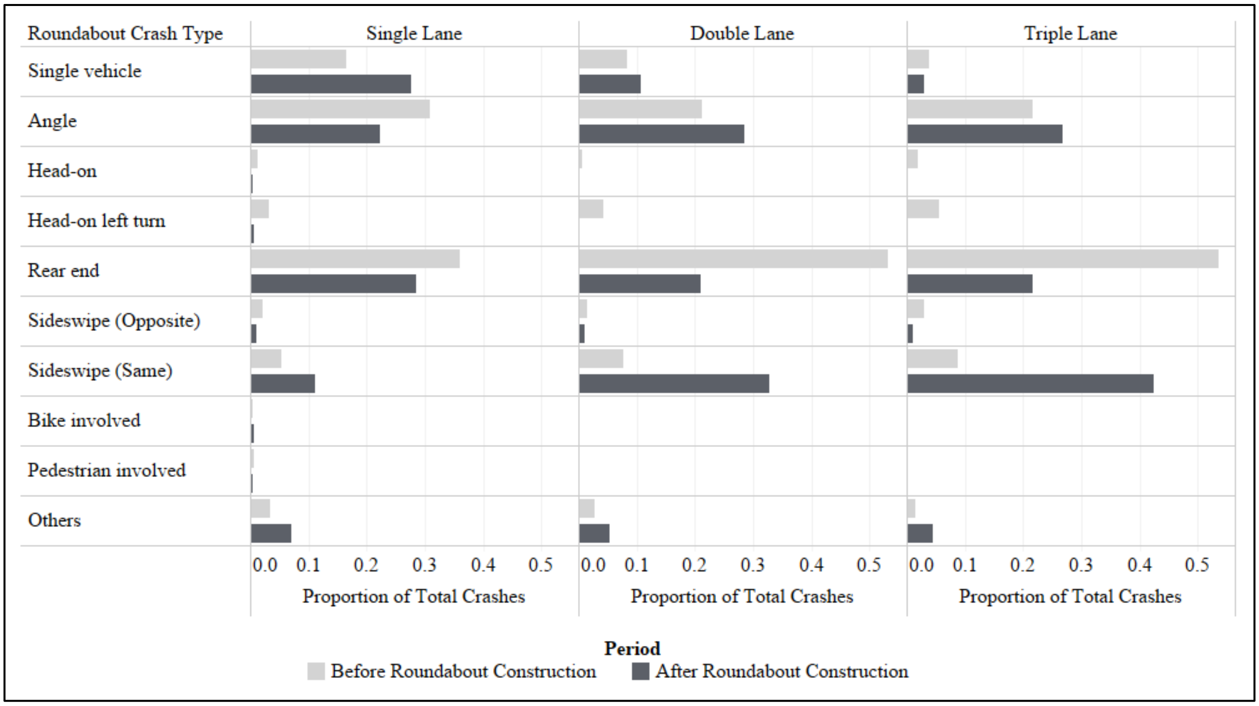

Figure 22 Changes in Proportion of Roundabout Crash Types Before and After Construction

4.2.4 Calculating Safety Benefits using Raw Crash Data

The simplest method to investigate the safety benefits of converting a conventional intersection to a roundabout would be to compare the annual crash frequencies before and after the construction of roundabout. This is also called naïve before-after study since the crashes from the pre-conversion period are used to predict crashes during the after period.

Let $N_{observed,B}$ and $N_{observed,A}$ be the number of observed crashes in the before and after period, respectively at each of the roundabout sites. The expected number of crashes in the after period, $N_{expected,A}$, are assumed to be equal to $N_{observed,B}$. The odds ratio (OR), also called crash modification factors (CMF) for each site is calculated as shown below. CMFs are multiplicative factors that are a measure of safety effectiveness of converting a stop-controlled or signalized intersection to a roundabout. These factors are used to compute expected number of crashes at a site after the treatment is installed (conversion to roundabout in the present study).

$$OR = \frac{N_{observed,A}}{N_{expected,A}} \qquad \text{Equation 11}$$

For analysis purposes, the before and after annual crash frequencies were determined. For 6 roundabouts out of 180, the construction year was unknown and hence were removed from this analysis. Also, roundabouts constructed before in 2004 or before, or in 2019 or after were excluded from this analysis since the crash data were available from 2004 to 2019. The final dataset included 142 roundabouts. Table 24 shows the results of the naïve before-after analysis along with CMF. The results show that the total crashes increased by 8%, 123%, and 233% after conversion to roundabouts on single lane, double lane, and triple lane roundabouts, respectively. Injury crashes reduced by 34% and 12% on single and double lane roundabouts, respectively, while it increased by 40% on triple lane roundabouts. It should be noted that these estimates are not controlled for differences in driver behavior, randomness, traffic volume, etc.

Table 24 Naive Before and After Analysis Results

Crash Severity	Roundabout Type	Number of Sites	CMF	Standard Error	Significant (5% Level)
Total	Single lane	92	1.08	0.03	Yes
	Double lane	42	2.23	0.05	Yes
	Triple lane	8	3.33	0.09	Yes
Injury	Single lane	92	0.66	0.51	Yes
	Double lane	42	0.88	0.06	Yes
	Triple lane	8	1.40	0.10	Yes

The results from Table 22 showed that the annual average crash frequency for total crashes shows a reduction in the after period on single lane roundabouts, which indicates a CMF of less than 1. However, the naïve before-after results from Table 24 shows a CMF of 1.08 for single lane roundabouts. This difference is due to the manner in which the statistics are calculated. The annual average crash frequency in Table 22 is calculated simply by taking the average of observed crash counts across all sites. The number of years in the before and after periods are different for each site. However, in the naïve before-after study, the average crash frequency in the before period is converted to the expected number of crashes in the after period for the same number of years in the after period. A smaller sample of before-period data may result in overestimating or underestimating the crash frequencies in the after period.

4.3 Empirical Bayes Safety Analysis

The empirical Bayes or EB method combines a site's observed and predicted crash frequencies based on a safety performance function (SPF) to estimate expected crash frequency had the treatment (roundabout conversion) not been implemented. For EB analysis, following terms need to be explained:

- The variable π is defined as the expected number of crashes at a specific site in the after period if the treatment has not been implemented. This variable only applies for the targeted crashes (i.e., total, single vehicle, etc.) and/or their severity (i.e., fatal, incapacitating injury, property damage only, etc.). π is referred to as the 'predicted value'.

- The variable λ is used to define the expected number of crashes in the after period (after the implementation of the treatment). λ is referred to as the 'estimated value'.

The effects of a treatment are estimated by comparing both variables above in the following manner:

- The reduction (or increase) in the expected number of crashes is given as $\delta = \pi - \lambda$. A positive number indicates a decrease in the expected number of crashes.
- The ratio or the Index of Safety Effectiveness is defined as $\theta = {}^{\lambda}/_{\pi}$. If the number of crashes analyzed is below 500 for the before period, θ needs to be adjusted by the following factor: $1 + Var\{\pi\}/\pi^2$. This adjustment is used to minimize the bias caused by a small sample size. The Index of Safety Effectiveness therefore becomes as shown in Equation 12. A value below 1.0 indicates a reduction in the number of crashes.

$$\theta = \frac{\lambda/\pi}{1+Var\{\pi\}/\pi^2} \qquad \text{Equation 12}$$

Where,

$Var\{\pi\}$ = variance of π,

$Var\{\lambda\}$ = variance λ.

The variance is a measure of uncertainty associated with the estimated value. The variance of the reduction, δ, is calculated as shown in Equation 13. The variance of the Index of Safety Effectiveness is calculated as shown in Equation 14.

$$Var\{\delta\} = Var\{\pi\} + Var\{\lambda\} \qquad \text{Equation 13}$$

$$Var\{\theta\} = \theta^2 \left[\frac{\left(Var\{\lambda\}/\lambda^2\right)+\left(Var\{\pi\}/\pi^2\right)}{(1+Var\{\pi\}/\pi^2)^2}\right] \qquad \text{Equation 14}$$

Table 25 lists the variables used when a reference/control group is utilized. The Latin characters represent the number of crashes that occurred at the sites under study. The Greek letters represent the expected or estimated number of crashes at those sites. How these variables are used is described below.

Table 25 Observed and Expected Number of Crashes

	Treatment Group	**Reference Group**
Before	K, κ	M, μ
After	L, λ	N, ν

The safety effectiveness of an intervention is estimated using a 4-step process (Hauer, 1997):

1. Estimate λ and π.
2. Calculate the variance of λ and π. As discussed above, they are defined as Var{λ} and Var{π}, respectively.
3. Estimate the difference δ and the Index θ.
4. Calculate the variance of δ and θ. They are defined as Var{δ} and Var{θ}, respectively.

The steps above are done for each site individually and the estimated and predicted values, as well as their variances, are summed for all the sites that are analyzed simultaneously.

The EB method allows the estimation of the safety benefits at treated sites using information from reference sites. The expected crash frequency ($E[\kappa|\mathrm{K}]$) at a treated site is a result of the combination of the predicted crash count *(E[κ])* based on the reference sites with similar traits and the crash history (*K*) of that site (usually during the before time period of the treated sites). It should be noted that the terms *κ* and *(E[κ])* are technically the same, but the latter is usually used for statistical models. Hence, for the EB method, *(E[κ])* is used rather than κ. The expected crash frequency and its variance are shown in Equations 15 and 16, respectively.

$E[\kappa|K] = w.E[\kappa] + (1 - w).K$ Equation 15

$Var[\kappa|K] = (1 - w).E[\kappa|K]$ Equation 16

Where,

w = weight factor between 0 and 1.

The parameter E[κ] is estimated from the safety performance functions (SPFs) developed using a negative binomial (NB) regression (also known as Poisson-gamma) model under the assumption that the covariates in the SPFs represent the main safety traits of the reference sites. The step by step procedure for using the before-after study with the EB method is described below.

Step 1. Develop Safety Performance Functions

Using crash, traffic, and geometric data from the reference sites, develop SPFs using regression models for all crashes, as well as crashes for various subsets of interest (e.g., fatal and injury). Since crashes are non-negative discrete integers, count data models such as negative binomial (NB) regression model were developed. This model is preferred over other count data models such as Poisson models since NB models accounts for overdispersion generally found in the crash data which refers to a phenomenon where the variance of the data is greater than the conditional mean. The NB regression model has the following modeling structure: the number of crashes Y_{it} for a particular i^{th} site and time period t when conditional on its mean $E[\kappa]_{it}$ is Poisson distributed and independent over all sites and time periods.

$$Y_{it}|E[\kappa]_{it} \sim Poisson(E[\kappa]_{it}),\ i = 1, 2, \ldots, i \text{ and } t = 1, 2, \ldots, t \qquad \text{Equation 17}$$

The mean of the Poisson is structured as:

$$E[\kappa]_{it} = f(X;\beta)exp(e_{it}) \qquad \text{Equation 18}$$

Where,

$f(.)$ is a function of the explanatory variables (X);

β is a vector of unknown coefficients; and,

e_{it} is the model error independent of all the covariates

Also, to remove any correlations among crash count observations across different years, the data were aggregated across years for each site and natural logarithm of number of years of data was used as offset variable in the model. The SPFs developed in this study are presented in subsequent section.

Step 2. Estimate the expected number of crashes in the before period

Using the SPFs developed in Step 1, estimate the expected number of crashes ($E[\kappa]_i$) for the before period at each treatment site. Obtain an EB estimate of the expected number of crashes ($E[\hat{\kappa}_i|K_i]$) before implementation of the countermeasure at each treatment site and an estimate of variance of $E[\hat{\kappa}_i|K_i]$. The "^" refers to an estimate of a variable.

The estimate $E[\hat{\kappa}_i|K_i]$ is given by combining the SPF predictions for the before period ($E[\kappa]_i$) with the total count of crashes during the before period (K_i) as follows:

$$E[\hat{\kappa}_i|K_i] = \widehat{w}_i \cdot E[\hat{\kappa}_i] + (1 - \widehat{w}_i) \cdot K_i \qquad \text{Equation 19}$$

The weight $\widehat{w}_i$ is given as shown in Equation 21.

$$\widehat{w}_i = \frac{1}{1+\alpha E[\widehat{\kappa}_i]} \qquad \text{Equation 20}$$

Where,

α is the inverse dispersion parameter of a NB regression model ($Var[Y_i] = E[\kappa_i] + \alpha E[\kappa_i]^2$).

The variance of the estimate is given as

$$Var[E[\hat{\kappa}_i|K_i]] = (1 - \widehat{w}_i) \cdot E[\hat{\kappa}_i|K_i] \qquad \text{Equation 21}$$

Step 3. Calculate the proportion of the after period crash estimate to the before period estimate

Using the SPFs developed in Step 1, estimate the expected number of crashes ($E[z]_i$) in the after period at each treatment site. The ratio between the after period crash estimate and the before period estimate (P_i) is calculated as

$$P_i = \frac{E[\hat{z}]_i}{E[\widehat{\kappa}]_i} \qquad \text{Equation 22}$$

Step 4. Obtain the predicted crashes $\hat{\pi}_i$ and its estimated variance

Calculate the predicted crashes during the after period that would have occurred without implementing the countermeasure (i.e., roundabout construction). The predicted crashes ($\hat{\pi}_i$) are given by:

$$\hat{\pi}_i = P_i \times E[\hat{\kappa}_i|K_i] \qquad \text{Equation 23}$$

The estimated variance of $\hat{\pi}_i$ is given by:

$$Var[\hat{\pi}_i] = {P_i}^2 Var[E[\hat{\kappa}_i|K_i]] = {P_i}^2 (1 - \widehat{w}_i) \cdot E[\hat{\kappa}_i|K_i] \qquad \text{Equation 24}$$

Step 5. Compute the sum of the predicted and observed crashes over all sites in the treatment group

The after-period crashes and their variances for a group of sites had the treatment not been implemented at the treated sites is given by:

$$\hat{\pi} = \sum_{i=1}^{j} \hat{\pi}_i \qquad \text{Equation 25}$$

Where,

j represents the total number of sites in the treatment group,

$\hat{\pi}$ is the expected after-period crashes at all treated sites had there been no treatment.

For a treated site, the crashes in the after-period are influenced by the implementation of the treatment. The safety effectiveness of a treatment is known by comparing the actual crashes with the treatment to the expected crashes without the treatment. The number of after-period crashes for a group of treated sites is given as:

$$\hat{\lambda} = \sum_{i=1}^{j} L_i \qquad \text{Equation 26}$$

Where,

L_i is the crash frequency during the after period at site *i*.

The estimate of $\hat{\lambda}$ is equal to the sum of the observed number of crashes at all treated sites during the after-study period.

Step 6. Estimate $Var[\hat{\lambda}]$ and $Var[\hat{\pi}]$

Based on the assumption of a Poisson distribution, the estimate of variance of $\hat{\lambda}$ is assumed to be equal to L. The estimate of variance of $\hat{\pi}$ can be calculated from the equation as follows:

$$Var[\hat{\lambda}_i] = L_i \qquad \text{Equation 27}$$

$$Var[\hat{\lambda}] = \sum_{i=1}^{j} Var[\hat{\lambda}_i] \qquad \text{Equation 28}$$

$$Var[\hat{\pi}_i] = (1 - \hat{w}_i) \cdot E[\hat{\kappa}_i | K_i] = (1 - \hat{w}_i) \cdot \hat{\pi}_i \qquad \text{Equation 29}$$

$$Var[\hat{\pi}] = \sum_{i=1}^{j} Var[\hat{\pi}_i] \qquad \text{Equation 30}$$

Step 7. Estimate $\hat{\delta}$ and $\hat{\theta}$

The ‘change in the safety’ (δ) and ‘index of safety effectiveness’ (θ) are calculated as described above:

$$\hat{\delta} = \hat{\pi} - \hat{\lambda} \quad \text{Equation 31}$$

$$\hat{\theta} = \frac{\left(\frac{\hat{\lambda}}{\hat{\pi}}\right)}{\left(1+\frac{Var(\hat{\pi})}{\hat{\pi}^2}\right)} \quad \text{Equation 32}$$

If $\hat{\delta}$ is greater than zero and $\hat{\theta}$ is less than one, then the treatment has a positive safety effect. In addition, the percent decrease in the number of target crashes due to the treatment is calculated as $100\left(1-\hat{\theta}\right)\%$.

Step 8. Estimate $Var[\hat{\delta}]$ and $Var[\hat{\theta}]$

The estimated variance and standard error of the estimated safety-effectiveness are given by:

$$Var\left[\hat{\delta}\right] = \hat{\pi} + \hat{\lambda} \quad \text{Equation 33}$$

$$Var\left[\hat{\theta}\right] = \frac{\hat{\theta}^2 \cdot \left[\frac{Var(\hat{\lambda})}{\hat{\lambda}^2} + \frac{Var(\hat{\pi})}{\hat{\pi}^2}\right]}{\left[1+\frac{Var(\hat{\pi})}{\hat{\pi}^2}\right]^2} \quad \text{Equation 34}$$

$$s.e[\hat{\theta}] = \sqrt{Var[\hat{\theta}]} \quad \text{Equation 35}$$

The 95% confidence interval for $\hat{\theta}$ is calculated as $\hat{\theta} \pm 1.96 s.e\left[\hat{\theta}\right]$. If the confidence interval contains the value one, then no significant effect has been observed at the 5% significance level.

4.3.1 Development of SPFs for Control Sites

For SPF development, the control sites were grouped together based on number of approach legs (three vs four) and type of traffic control (signalized vs stop-controlled). Initially, separate SPFs were developed for each of these 4 categories. However, due to a lower sample size of three-legged intersections, SPFs were developed based on type of control, i.e., SPFs were developed for signalized intersections and stop-controlled intersections. The research team first examined different functional forms with various combinations of variables while modeling the crashes. This included several variables such as traffic volume, number of approach lanes, speed limit, width of lanes, roadway context, whether intersection is at an interchange or not, etc. However, most of these variables were not found to be statistically insignificant at 95% confidence. The form presented below reflects the findings from several preliminary regression analyses. The same form was also used for modeling the fatal and injury (FI) crashes. Note that the designation *i* and *t* are removed to simplify the description of the results. The predicted crash frequency for signalized intersection and stop-controlled intersection is calculated using Equation 36 below.

$$E[k]\ per\ year = e^{\beta_0+\beta_1 I_{3-leg}+\beta_2 I_{Major\ PSL}+\beta_3 I_{Lane\ count}+\beta_4 I_{Minor\ lane\ width}+\beta_5 I_{Interchange}} \times (TEV)^{\beta_6}$$

Equation 36

Where,

TEV = total entering volume (vehicles per day)

$\mathrm{I}_{3-\mathrm{leg}}$ = Indicator variable three-legged intersection (0 for four-legged intersection)

$\mathrm{I}_{\mathrm{Major\ PSL}}$ = Indicator variable that is 1 if major posted speed limit is 25 mph or less

$\mathrm{I}_{\mathrm{Lane\ count}}$ = Indicator variable that is 1 if number of lanes on major approach is 3 or more

$\mathrm{I}_{\mathrm{Minor\ lane\ width}}$ = Indicator variable that is 1 if lane width on minor approach is 12 ft or more

$\mathrm{I}_{\mathrm{Interchange}}$ = Indicator variable that is 1 if intersection is located on interchange

Table 26 shows the calibrated coefficients for total crashes and FI crashes using the control sites database. While calibrating the models, sites with unknown AADT data were removed.

Table 26 Safety Performance Function for Intersections

Model	Number of Sites	β_0	β_1	β_2	β_3	β_4	β_5	β_6	Overdispersion Parameter (k)
Total Crashes									
Signalized Intersection	37	-4.796	na	-0.756	na	na	-0.863	0.768	0.536
Stop-Controlled Intersection	57	-8.383	-0.412	na	na	-0.239	na	1.063	0.169
Fatal and Injury (KABC) Crashes									
Signalized Intersection	37	-6.645	na	na	0.610	na	-1.079	0.755	0.403
Stop-Controlled Intersection	57	-8.349	-0.506	-0.471	0.503	-0.254	na	0.895	0.115

4.3.2 EB Analysis Results

Using the procedure outlined earlier, the EB analyses were carried out for various combinations of roundabout categories. For all EB analyses purposes, roundabout sites with 5 or 6 legs, sites with unknown AADT information, sites with unknown construction year or the ones constructed outside the analysis period of 2004-2019 were excluded. Additionally, there were few sites where it was not possible to determine the type of traffic control that was present prior to roundabout construction, or the roundabout was constructed on a new roadway intersection. Such sites were also excluded from the EB analyses thereby reducing the number of roundabouts from 180 to 97. Table 27 summarizes the results of the EB analyses. For comparison purposes, CMFs from the MDOT 2011 study on roundabouts are also provided (Bagdade et al., 2011). The results show that the total crashes increased while FI crashes generally reduced due to conversion of conventional intersection to roundabout across all categories. The only exception was triple lane roundabouts which showed an increase in FI crashes too, although the result was not statistically significant. This is expected since reducing crash severity at intersections is one of the primary goals of converting an intersection to a roundabout.

Table 27 Summary of EB Analyses Results

Category	Sites	Crash Severity	CMF (θ)	Std. Error	Significant (5% Level)	Current MDOT CMF
All sites	97	Total	2.10	0.050	Yes	1.35
		FI	0.92	0.047	No	0.58
Roundabouts not on state-maintained roads	78	Total	2.15	0.053	Yes	-
		FI	0.95	0.051	No	-
Roundabouts on state-maintained roads	19	Total	2.03	0.094	Yes	-
		FI	0.91	0.090	No	-
Sites on interchange	20	Total	1.28	0.066	Yes	-
		FI	0.67	0.075	Yes	-
Sites not on interchange	77	Total	2.33	0.057	Yes	-
		FI	1.00	0.053	No	-
Sites on interchange (previously stop-controlled)	11	Total	1.35	0.084	Yes	1.25
		FI	0.66	0.089	Yes	0.42
Sites on interchange (previously signalized)	9	Total	1.17	0.104	No	-
		FI	0.69	0.134	Yes	-
Sites on interchange (Minus triple lane)	18	Total	1.21	0.063	Yes	-
		FI	0.74	0.085	Yes	-
Sites not on interchange (Minus triple lane)	71	Total	1.83	0.049	Yes	-
		FI	0.81	0.050	Yes	-
Single lane roundabouts	58	Total	1.03	0.044	No	-
		FI	0.60	0.055	Yes	-
Double lane roundabouts	31	Total	2.13	0.060	Yes	-
		FI	0.94	0.064	No	-
Triple lane roundabouts	8	Total	3.07	0.145	Yes	-
		FI	1.26	0.120	Yes	-
Single and double lane roundabouts	89	Total	1.68	0.040	Yes	1.00
		FI	0.79	0.043	Yes	0.49
Signalized intersection to roundabout	47	Total	2.05	0.054	Yes	-
		FI	0.90	0.054	No	-
Stop controlled intersection to roundabout	50	Total	2.27	0.091	Yes	1.03
		FI	1.04	0.079	No	0.64
Signalized intersection to roundabout (minus triple lane)	43	Total	1.92	0.058	Yes	0.78
		FI	0.81	0.057	Yes	0.30
Stop controlled intersection to roundabout (minus triple lane)	46	Total	1.29	0.051	Yes	-
		FI	0.76	0.064	Yes	-
Signalized intersection to triple lane roundabout	4	Total	2.28	0.112	Yes	1.98
		FI	1.07	0.122	No	0.80

4.4 Cross-Sectional Safety Analysis

Cross-sectional analysis uses statistical modeling techniques that consider crash experience of sites of interest with and without a particular treatment of interest (conversion of a roundabout in this study). Cross-sectional studies are used in situations where before-after evaluation is not feasible due to lack of pre-treatment data, or if the treatment installation dates are not known. In such cases, the cross-sectional safety evaluation method can be used to estimate the safety effectiveness of the treatment through comparison to crash data at comparable non-treatment sites.

Although in this study the before-after data were collected, cross-sectional analyses were still done to include roundabouts with unknown construction year, or with missing before-construction data. To that end, data for all the control sites from 2004 to 2019, and data for all the roundabouts after their respective construction year were compiled. The data were arranged at site-year level with each row representing one year of data for one site (roundabout or control site). Sites with unknown AADT information were removed. Roundabouts with more than 4 approach legs were also removed from the analysis. This resulted in total 108 roundabouts and 94 control sites being included in cross-sectional analysis.

To compare the safety effectiveness of the roundabouts through cross-sectional analysis, NB statistical modeling technique was used that included development of a single model for all the data which includes a binary indicator to indicate whether the site is a roundabout or not. NB models were developed separately for total crashes and fatal and injury (KABC) crashes. Table 28 and Table 29 present the results of the NB regression models for total crashes and fatal and injury crashes, respectively.

Table 28 Random Effects NB Model for Total Crashes using Cross-Sectional Data

Parameter	Estimate	Std. Error	p-value
Intercept	-8.373	0.644	<0.001
Site Type			
Traditional intersection		*Baseline*	
Roundabout	0.459	0.118	<0.001
Ln (TEV)	1.028	0.073	<0.001
Number of Approach Legs			
Four		*Baseline*	
Three	-0.315	0.116	0.007
Number of Lanes on Major Road			
1		*Baseline*	
2	0.508	0.124	<0.001
3 or more	0.671	0.191	<0.001
Site on Interchange			
No		*Baseline*	
Yes	-0.281	0.124	0.024
Variance of Intercept	0.400	0.050	<0.001
Overdispersion Parameter	0.532		

Table 29 Random Effects NB Model for FI Crashes using Cross-Sectional Data

Parameter	Estimate	Std. Error	p-value
Intercept	-9.590	0.809	<0.001
Site Type			
Traditional intersection		*Baseline*	
Roundabout	-0.309	0.132	0.020
Ln (TEV)	1.001	0.091	<0.001
Number of Approach Legs			
Four		*Baseline*	
Three	-0.317	0.129	0.014
Number of Lanes on Major Road			
1		*Baseline*	
2	0.395	0.135	0.004
3 or more	0.536	0.204	0.009
Site on Interchange			
No		*Baseline*	
Yes	-0.281	0.135	0.037
Variance of Intercept	0.372	0.057	<0.001
Overdispersion Parameter	0.562		

The results show that the parameter estimates for binary indicator for type of site (1 for roundabout, and 0 for traditional intersection) is positive for total crashes and negative for FI crashes. This indicates that the average number of total crashes is higher at roundabouts compared to traditional signalized or stop-controlled intersections. Similarly, fatal and injury crashes are lower on roundabouts. The parameter coefficients can be exponentiated to get CMF as presented in Table 30. The CMF of 1.58 and 0.73 for total crashes and FI crashes, respectively, indicate that converting a stop-controlled or signalized intersection to a roundabout would increase total crashes by 58% and reduce fatal and injury crashes by 27%, on average.

Table 30 CMF for Converting an Intersection to Roundabout based on Cross-Sectional Analysis

Crash Severity	Parameter Estimate from Cross-Sectional Analysis	CMF
Total crashes	0.459	1.58
FI crashes	-0.309	0.73

4.5 Development of Safety Performance Function for Roundabouts

As a part of the safety analyses, SPFs were also developed for roundabouts. These SPFs can be used to evaluate safety performance of roundabouts. Separate SPFs were developed for total crashes and fatal and injury (FI) crashes. Initially, roundabouts were grouped together based on their geometry including number of approach legs, number of circulating lanes, roadway context, whether the roundabout is located on an interchange, etc. However, to ensure sufficient sample size in each category, certain categories were grouped together. Also, roundabouts with unknown AADT data, or with more than 4 legs were removed. Triple lane roundabouts were also not included in SPF development due to their very low sample size and significantly different operational conditions compared to single and double lane roundabouts. After several iterations of modeling, the final SPFs were developed for three groups-

- Four-legged roundabout with single circulating lane (48 sites)
- Four-legged roundabout with two circulating lanes (25 sites)
- Three-legged roundabout (27 sites)

To develop the SPFs, the multiple years of data were aggregated and number of years of data for each roundabout was determined. The crash data was modeled using negative binomial regression with natural log of number of years of data as the offset variable. This would allow us to ignore the random effects framework since there will be no repetition in the data. The response variable in this model specification will be crashes/year/site.

In selecting the final SPFs, the statistical significance of included parameters and goodness of fit of the overall model were considered. Table 31, Table 32 and Table 33 present the final SPFs for four-legged single lane, four-legged multilane, and three-legged roundabouts, respectively. The tables present estimated parameter coefficients along with standard error in parenthesis. Significance level is also denoted by asterisks (a single asterisk represents a parameter significant at $\alpha = 0.10$, double asterisks correspond to significance at $\alpha = 0.05$). It should be noted that several other geometric variables were also considered in the analysis, including the number of approach lanes, number of access points within 250 ft of the roundabout yield line, width of circulating lanes, exit width, presence of right turn bypass lane, and diameter of central island. However, the effect of these variables was either statistically insignificant or they exhibited a significant degree of correlation with other geometry-related variables, and hence, were not included in the final SPF.

Table 31 SPF for Four-Legged Single Lane Roundabouts

Parameter	**Total Crashes**	**FI Crashes**
	Estimate (Std. Error)	**Estimate (Std. Error)**
Intercept	-7.297 (1.152)**	-11.330 (1.706)**
Ln (TEV)	1.093 (0.136)**	1.443 (0.202)**
Maximum inscribed diamter (ft)	-0.010 (0.004)**	-0.019 (0.005)**
Context		
Rural	*Baseline*	
Urban	na	-0.658 (0.216)**
Site on interchange		
No	*Baseline*	
Yes	-0.623 (0.221)**	n/a
Overdispersion Parameter	0.219	0.066

Table 32 SPF for Four-Legged Double Lane Roundabouts

Parameter	Total Crashes	FI Crashes
	Estimate (Std. Error)	**Estimate (Std. Error)**
Intercept	-10.856 (2.669)**	-4.496 (4.154)
Ln (TEV)	1.096 (0.259)**	0.540 (0.408)
Maximum entry width (ft)	0.081 (0.039)**	n/a
Speed Limit on Minor Road		
> 25 mph	*Baseline*	
<= 25 mph	na	-0.956 (0.487)*
Context		
Rural	*Baseline*	
Urban	0.897 (0.039)**	na
Overdispersion Parameter	0.490	0.646

Table 33 SPF for Three-Legged Roundabouts

Parameter	Total Crashes	FI Crashes
	Estimate (Std. Error)	**Estimate (Std. Error)**
Intercept	-5.854 (1.280)**	-9.293 (2.188)**
Ln (TEV)	0.775 (0.139)**	0.890 (0.231)**
Roundabout Type		
Single lane	*Baseline*	
Double lane	0.677 (0.218)**	0.682 (0.252)**
Speed Limit on Minor Road		
> 25 mph	*Baseline*	
<= 25 mph	-0.587 (0.264)**	na
Overdispersion Parameter	0.163	0.087

4.6 Summary

As a part of roundabout safety evaluation, a comprehensive list of all known roundabouts in Michigan was compiled. Suitable control sites for each roundabout with similar traffic volume and roadway characteristics were also identified. Relevant roadway geometry data, traffic volume data, and crash data were compiled and integrated. The safety analyses for roundabouts was done at various levels of detail. Starting with simple before-after comparison of average crash frequencies on each roundabout. This was followed by Empirical Bayes (EB) analysis that also considered the impacts of other important site characteristics. As a part of EB analysis, SPFs were also developed for total crashes and fatal and injury crashes for conventional signalized intersections and stop-controlled intersections. Cross-sectional analysis was also done which considered only after-period data for roundabout. Lastly, SPFs for roundabouts were also developed separately for total crashes and fatal and injury crashes for three-legged roundabouts, four-legged single lane roundabouts, and four-legged double lane roundabouts. Overall, the results showed that the total crashes increase as a result of roundabout construction while the fatal and injury crashes reduce significantly.

Naïve before-after analysis showed total crashes to increase by 8%, 123% and 133% on single lane, double lane, and triple lane roundabouts, respectively, while injury crashes reduced by 34%, 12% on single lane and double lane roundabouts, respectively. On triple lane roundabouts, injury crashes increased by 40%. Subsequent EB analysis that controlled for the effects of traffic volume and similar group of control sites showed that total crashes increased by 3%, 113%, and 207% on single lane, double lane, and triple lane roundabouts, respectively. FI crashes reduced by 40% and 6% on single lane and double lane roundabouts, respectively. Triple lane roundabouts showed an increase in FI crashes by 26%. Converting a signalized intersection to a roundabout with one or two circulating lanes was found to have a CMF of 1.92 and 0.81 for total crashes, and FI crashes respectively. Similarly, conversion of stop-controlled intersection to a roundabout (with 1 or 2 circulating lanes) had a CMF of 1.29 and 0.76 for total crashes and FI crashes, respectively. Lastly, cross-sectional study also showed that the total crashes on roundabouts are, on average, 58% more than the conventional intersections, while the fatal and injury crashes are 27% less on roundabouts.

5 LEVEL OF SERVICE, ENVIRONMENTAL AND ECONOMIC ANALYSES OF ROUNDABOUTS

5.1 Delay and Level of Service

The extant research literature has generally shown a significant reduction in delay as a result of converting conventional intersections to roundabouts as discussed in Chapter 2. The present study also compares the effect of such conversions on vehicle delay. To compare the control locations and their roundabout counterparts, various geometric and traffic parameters were collected. Traffic volume data was obtained from MDOT's Transportation Data Management System (TDMS) as discussed previously in the safety analyses chapter. Geometric data were collected from aerial and street-level imagery from Google Earth. This information included dimensions such as lane and opening widths, the angle between legs, etc.

Rodel Roundabout Analysis Software (Rodel) (*Rodel Interactive*, 2022) integrates both AASHTO Highway Capacity Manual model and lane-based geometric models and has been validated with U.S. data and is accurate for North American analyses. The current version of Rodel (v1.96) was used to calculate the Level of Service (LOS) and delay (seconds) for the roundabouts, based on the most recent major- and minor-road volumes available. While there are certain default parameters, Rodel requires geometric inputs related to the approach, entry, circulating, and exit parameters, along with bypass geometry is present. Peak hour flows were entered based on count data available in TDMS. These are all used by Rodel to calculate arrival flows, capacity, LOS, and delay.

From a traffic operations perspective, the LOS and delay are useful to compare different intersection types both qualitatively and quantitatively. In particular, the change in delay from a stop-controlled or signalized intersection to a roundabout is a useful metric for consideration when changing other intersections within Michigan.

Synchro v11 (Trafficware, 2022) was used to calculate the LOS and delay for the comparison sites (stop-controlled and signalized intersections). Synchro implements the methodologies detailed in the Highway Capacity Manual for the signalized and stop-controlled intersections used as comparison sites as well as for the roundabout locations in their prior condition.

While 15 roundabout/intersection pairs were used, this is only a sample of the total number of roundabouts within the state. Additionally, while similar locations were used for comparison (e.g. adjacent intersections along a corridor, similar traffic volumes, traffic control before conversion to a roundabout, etc.) the differences between locations do not result in a perfect 1:1 comparison. Table 34 identifies the roundabout locations, their paired control sites, the traffic control at the control site, the LOS, and delay (in seconds).

Recognizing there are differences in traffic volumes and number of lanes at each paired location, the average seconds of delay for the control sites is 16.7 seconds in the AM peak period, 16.3 seconds during off-peak times, and 29.9 seconds during the PM peak period. For the roundabouts, these times drop to 7.3, 5.3, and 10.1 seconds respectively. This translates to a 56.6% reduction in delay in the AM peak, 67.3% during off-peak periods, and a 66.1% reduction during the PM peak.

When focusing on urban locations, the AM peak, off-peak, and PM peak delays are reduced by 9.4, 12.0, and 21.8 seconds respectively; for rural locations these reductions are 9.5, 8.5, and 13.9 seconds.

The seconds of delay for a given letter grade (A-F) are not the same for roundabouts and stop-controlled or signalized intersections. When focusing on the prior traffic control, signalized intersections improve by 7.8 seconds in the AM peak, 11.2 seconds during off-peak periods, and 14.2 seconds during the PM peak. Rural locations experience improvements of 14.0, 10.8, and 34.7 seconds respectively.

While this implies that in general, conversion to a roundabout provides for reductions in delay, it should be noted that three of the fifteen locations did see an increase. All three of these locations were in urban locations and compared to their signalized counterparts. All the rural locations as well as all stop-controlled locations analyzed saw reductions in delay. The lowest-performing roundabout and period is the PM peak at the US-23 southbound off-ramp to Geddes Road, which experiences 38.4 seconds of delay during the PM peak. For the control locations, the PM peak at the US-23 southbound ramps to Silver Lake Road experience more than three times the delay at 116.8 seconds.

Table 34 Roundabout and Control Site LOS and Delay

	Roundabout	LOS (Delay) AM	LOS (Delay) OP	LOS (Delay) PM		Control Site	Control Site Control Type	LOS (Delay) AM	LOS (Delay) OP	LOS (Delay) PM
Roundabout	US-23 NB Off Ramp & Geddes Road	B (10.5)	A (6.7)	A (9.9)	Control Site	M-17 & US-23 NB	Signalized	A (0.4)	A (0.4)	A (0.6)
	US-23 SB Off Ramp & Geddes Road	C (21.4)	A (7.2)	E (38.4)		M-17 & US-23 SB	Signalized	A (0.7)	A (0.7)	A (0.6)
	US-23 SB & Lee Rd	A (4.4)	A (5.2)	A (8.7)		Silver Lake Rd & US-23 SB**	Stop Controlled	D (26.9)	C (20.5)	F (116.8)
	M-52 & Werkner Road	A (6.7)	A (4.8)	A (6.3)		Main St & Middle St	Signalized	B (13.5)	B (11.7)	C (21.4)
	US-127 BR & Mission Rd	A (5.3)	A (5.4)	A (6.0)		M-115 & Beaver Rd	Stop Controlled	B (11.2)	B (12.7)	B (13.3)
	M-5 & Pontiac Trail	A (4.8)	A (3.5)	A (7.4)		M-5 & Maple Rd*	Signalized	B (10.2)	A (8.5)	B (11.0)
	NB I-75 Off Ramp & Bristol Road	A (2.9)	A (2.8)	A (3.5)		Corunna Rd & I-75 NB	Signalized	B (18.4)	B (20.0)	D (54.3)
	I-94 EB Off Ramp & Sprinkle Road	A (4.0)	A (2.9)	A (4.5)		Portage Rd & I-94 EB	Signalized	B (10.1)	A (7.6)	B (10.1)
	I-94 WB Off Ramp & Sprinkle Road	A (6.3)	A (5.0)	A (7.4)		35th St & I-94 WB	Stop Controlled	B (12.1)	A (9.8)	B (11.7)
	US-41/M-28 & Front Street	A (4.0)	A (5.1)	A (6.7)		US-41 & Midway Dr	Stop Controlled	D (25.6)	C (20.8)	D (25.6)
	US-10 BR/ M-20 & Patrick Road	C (17.8)	A (8.8)	C (18.8)		Saginaw Rd & James Savage Rd	Signalized	B (12.0)	B (11.6)	B (11.7)
	I-75 NB off ramp & M-81/Washington Road	A (5.5)	A (5.5)	A (7.8)		Tittabawassee Rd & I-675 NB	Signalized	B (13.4)	A (9.9)	A (10.0)
	M-53 NB off ramp & 26-Mile	A (4.1)	A (4.6)	B (11.1)		M-53 NB Ramps & 23 Mile Rd*	Signalized	C (21.5)	C (20.0)	D (41.9)
	M-11 & Remembrance Road	A (5.4)	A (4.9)	A (7.2)		Kinney Ave & Remembrance Rd	Signalized	C (26.4)	D (37.3)	D (51.8)
	Farmington Rd & Maple Rd	A (6.5)	A (6.6)	A (9.9)		Orchard Lake Rd & Maple Rd	Signalized	D (48.7)	D (53.4)	E (67.9)
	US-10 WB @ M-30	A (6.6)	A (6.5)	A (8.6)		n/a				
						*Synchro reported delay/LOS used in place of HCM 6th Edition values due to incompatibilities with methodology ** Delay and LOS provided for stop-controlled movement only				

The LOS for signalized and stop-controlled is defined as a function of the average vehicle delay. When communicating to a non-technical audience, providing letter grades (where "A" is best) helps communicate the operations in a manner more easily understood than a numeric value. Converted the letter grades to an equivalent "grade point average" (where an "A" is a 4.0, "B" is a 3.0, etc.) the signalized and stop-controlled intersections get B's and C's while the roundabouts achieve A's. More specifically, for the non-roundabout locations, the average AM peak gets a 2.6 ("B-"), the off-peak a 2.9 ("B"), and the PM peak a 2.2 ("C"). Conversely, the roundabouts receive A's – a 3.7 ("A-") in the AM peak, a 4.0 ("A") during off-peak and a 3.6 ("A-") during the PM peak.

5.2 Environmental Analyses

The research literature has generally shown reduction in vehicle delays is generally associated with positive environmental benefits, in the form of reduced idling, reduced vehicle delays, and improved fuel consumption. While overall fuel economy and emissions have improved in the U.S. vehicle fleet, a large mix of vehicles on the road (both newer and older, gasoline and diesel vehicles, and various states of repair) will still allow roundabouts to highlight improvements. Additionally, while electric vehicles may not result in the same level of emissions from the vehicle itself, there are still particulate emissions generated by the contact of a vehicles tires on the roadway, and a reduction in stopping associated with roundabouts will help reduce this form or pollution (brake pad dust and rubber particulates from stopping friction).

While vehicles continually evolve and improve their emissions handling and fuel consumption, the mix of vehicles on our roadways will not change overnight. According to research by HIS Markit, the average age of vehicles in the U.S. increased to 12.2 years in 2022 (Parekh & Campau, 2022). It is unlikely that these older vehicles would be retrofitted with emissions or fuel reduction technologies so the benefits associated with operational improvements (e.g. roundabouts) will still be beneficial to both overall air quality and in particular in the immediate vicinity of the intersections.

The mix of vehicles in the U.S. is also changing. Comparing the same months of 2019 and 2022, according to information provided by JATO Dynamics, the U.S. vehicle market saw an increase in pickups and SUVs and a decrease in sedans, hatchbacks, and other body types (Munoz, 2022). Specifically, pickups and SUVS increased their market share by 9.9 percent during this period; with pickups rising from 16.9 percent to 19.4 percent, and SUVs increasing from 46.1 percent to 53.5 percent. Passenger vehicles and hatchbacks decreased from 22.4 percent and 4.9 percent to 16.3 percent and 3.8 percent over the same timeframe. While the average vehicle size has increased, this is generally offset by increased fuel economy and a growing number of alternative fuel vehicles (primarily battery electric).

The Argonne National Laboratory developed a calculation method for the savings associated with idling reduction (Argonne National Laboratory, n.d.). This considers a mix of vehicle classifications (Classes 1-8), vehicles sizes (e.g. small, medium, and large passenger cars), fuel types (both gasoline and diesel), engine sizes, and engine load (i.e. use of air conditioning). This information, alongside assumptions about the vehicle fleet on the roads in Michigan, can be used to estimate the fuel savings associated with delay reductions attributed to roundabouts.

The following assumptions were used in these calculations:

- An average of 5 percent of vehicles are commercial vehicles (classes 5-8)
    - Commercial vehicles use an average of 1.0 gal/hour while idling
- 95 percent of vehicles are passenger vehicles
    - Cars and hatchbacks account for 19.1 percent (16.3% + 3.8% = 20.1% x 0.95)
        - These classes of vehicles use an average of 0.25 gal/hour while idling
    - Pickups and SUVs account for 69.3 percent (16.9% + 53.5% = 72.9% x 0.95)
        - These classes of vehicles use an average of 0.50 gal/hour while idling
    - The remainder – 11.6 percent is undefined, though includes other body types such as sports cars, minivans, etc.
        - These are assumed to share similar numbers to cars and hatchbacks (0.25 gal/hour while idling)
- The peak hours (AM and PM) each represent 8% of the daily traffic.

Table 35 shows the list of control sites and their AADT information. Using the change in delay from Table 34, the traffic volumes from Table 35, and the assumed 8% peak hour factor, the daily delay for each location is calculated from the equation below in Table 36.

$$\Delta_{delay,TOT} = \frac{[phf_{AM} \times ADT \times \Delta_{delay,AM} + phf_{pM} \times ADT \times \Delta_{delay,PM} + (1 - phf_{AM} - phf_{PM}) \times ADT \times \Delta_{delay,OP}]}{3{,}600\ s/hr} \quad \text{Equation 37}$$

Where:

$\Delta_{delay,TOT}$ = total daily delay (hours)

$\Delta_{delay,AM}$ = AM peak hour delay (seconds)

$\Delta_{delay,PM}$ = PM peak hour delay (seconds)

$\Delta_{delay,OP}$ = delay in off peak hours (seconds)

phf_{AM} = AM peak hour factor (proportion of daily traffic in AM peak hour)

phf_{PM} = PM peak hour factor (proportion of daily traffic in AM peak hour)

ADT = average daily traffic

Table 35 Control Site Traffic Volume

Control Site	AADT Major	AADT Major	AADT Total
M-17 & US-23 NB	18,151	5,237	23,388
M-17 & US-23 SB	18,151	4,697	22,848
Silver Lake Rd & US-23 SB	17,960	4,225	22,185
Main St & Middle St	11,411	5,491	16,902
M-115 & Beaver Rd	3,578	6,069	9,647
M-5 & Maple Rd	30,546	24,315	54,861
Corunna Rd & I-75 NB	24,380	2,399	26,779
Portage Rd & I-94 EB	31,928	8,439	40,367
35th St & I-94 WB	31,928	3,325	35,253
US-41 & Midway Dr	21,547	14,929	36,476
Saginaw Rd & James Savage Rd	7,535	3,314	10,849
Tittabawassee Rd & I-675 NB	15,571	4,298	19,869
M-53 NB Ramps & 23 Mile Rd	57,454	5,361	62,815
Kinney Ave & Remembrance Rd	12,549	7,044	19,593
Orchard Lake Rd & Maple Rd	18,426	17,607	36,033

Table 36 Daily Delay Savings After Roundabout Conversion

Control Site	AM Peak Hour Delay Savings (s)	Off Peak Delay Savings (s)*	PM Peak Hour Delay Savings (s)	Total Daily Delay Savings (h)
M-17 & US-23 NB	-18,898	-123,769	-17,401	-44
M-17 & US-23 SB	-37,836	-124,750	-69,092	-64
Silver Lake Rd & US-23 SB	39,933	285,122	191,856	144
Main St & Middle St	9,195	97,964	20,418	35
M-115 & Beaver Rd	4,553	59,155	5,634	19
M-5 & Maple Rd	23,700	230,416	15,800	75
Corunna Rd & I-75 NB	33,206	386,903	108,830	147
Portage Rd & I-94 EB	19,699	159,369	18,084	55
35th St & I-94 WB	16,357	142,140	12,127	47
US-41 & Midway Dr	63,031	481,045	55,152	166
Saginaw Rd & James Savage Rd	-5,034	25,517	-6,162	4
Tittabawassee Rd & I-675 NB	12,557	73,436	3,497	25
M-53 NB Ramps & 23 Mile Rd	87,438	812,575	154,776	293
Kinney Ave & Remembrance Rd	32,916	533,243	69,908	177
Orchard Lake Rd & Maple Rd	121,647	1,416,529	167,193	474

On a per-vehicle basis, this equates to an average of 12.76 seconds of delay saved per vehicle, which when multiplied by an intersection's AADT could be used for planning purposes to estimate the delay savings. In other words, an intersection with an AADT of 10,000 veh/day could assume to have a combined reduction of 127,600 seconds in delay, which is 35.44 hours/day or 12,937.22 hours/year.

Using the mix of vehicles assumed above along with the fuel consumption used when idling, the 15 evaluation sites combine for a daily fuel savings of 735 gallons, or 49 gallons/day/intersection. This can also be equated to 0.001678 gal/veh/day based on the traffic volumes for the 15 sites. At a current average of $3.75/gallon for gasoline, this represents a savings of $183.75 per location/day, or $67,068.75 per year. On a per vehicle basis, this equates to a savings of $2.30 per vehicle/year. For planning purposes, using the example above, an intersection with 10,000 AADT could represent $23,000 in annual fuel savings.

Using the user delay costs published by MDOT, cars have a user cost per hour of $19.66 and trucks at $34.68 (Michigan Department of Transportation, 2022). These values can also be used to calculate the user delay savings for roundabouts (assuming a 5% commercial vehicle percentage). On a per-vehicle basis, this represents a yearly savings of $66.80 per vehicle. Again, for planning purposes, an intersection with 10,000 AADT could represent $668,000 in annual delay savings after conversion to a roundabout. The savings above are summarized in Table 37.

Table 37 Annual Savings per Vehicle

Measure	Annual Savings per Vehicle
Delay	1.2939 hours
Delay costs	$66.80
Fuel	0.612344 gallons
Fuel costs	$2.30 (@3.75/gallon)

5.3 Economic Analysis

Cost-benefit analysis was conducted to assess whether the benefits of converting an intersection to a roundabout outweighs the costs of construction. The analyses were done for 15 roundabout/intersection pairs identified previously for level of service and environmental analysis as listed in Table 34. Only tangible costs and benefits/dis-benefits directly resulting from roundabout conversion were considered including:

- Road user benefit/dis-benefit
    - Traffic costs
    - Delay
    - Fuel consumption
- Agency costs which include the cost of converting a traditional intersection into a roundabout

Since the roundabout/intersection pairs identified in Table 34 were carefully identified in such a way that the intersections represent the control site for their respective roundabout site, it is reasonable to assume that the control sites represent the traffic conditions at roundabout locations before the conversion. Thus, the delay and fuel savings for each of these 15 roundabouts are determined based on the savings calculated above for each of their respective control sites.

5.3.1 Road User Benefits/Dis-Benefits

The level of service and environmental analyses above showed that roundabout conversion generally resulted in reduced delay and reduced fuel consumption, which can be considered as road user benefits. Total crashes generally increased after roundabout conversions, but fatal and injury crashes were reduced. The changes in associated crash costs before and after the conversion can be considered as another road user benefit/dis-benefit. The following provide separate details for these (dis-)benefits.

5.3.1.1 Delay Costs

Table 36 shows the savings in delay associated with each of the 15 roundabout/intersection pairs. As stated earlier, user delay costs per hour were $19.66 for passenger cars and $34.68 for trucks (Michigan Department of Transportation, 2022). By assuming a 5% truck volume at each of these sites and using the traffic volume from Table 35**,** Table 38 shows the monetary savings associated with delay reductions at each of the sites and also combined for all sites. All except two sites showed savings in terms of reduced delay due to roundabout conversions. Both of the sites are located on US-23 off ramp and Geddes Road and in close proximity. Both of these sites are in urban areas.

Table 38 Estimated Delay Savings Associated with Roundabout Conversion

Roundabout Site	Total Daily Delay Savings (h)	Annual Delay Savings per Vehicle (h)	Annual Delay Savings for Trucks ($)	Annual Delay Savings for Cars ($)	Total Annual Delay Savings ($)
US-23 NB Off Ramp & Geddes Road	-44	-0.694	(28,141.29)	(319,064.43)	(347,205.72)
US-23 SB Off Ramp & Geddes Road	-64	-1.028	(40,730.92)	(461,805.04)	(502,535.97)
US-23 SB & Lee Rd	144	2.362	90,877.26	1,030,361.57	1,121,238.83
M-52 & Werkner Road	35	0.765	22,429.10	254,299.94	276,729.04
US-127 BR & Mission Rd	19	0.729	12,190.90	138,219.79	150,410.70
M-5 & Pontiac Trail	75	0.499	47,453.48	538,025.06	585,478.54
NB I-75 Off Ramp & Bristol Road	147	2.003	92,991.88	1,054,337.05	1,147,328.93
I-94 EB Off Ramp & Sprinkle Road	55	0.495	34,660.96	392,984.18	427,645.14
I-94 WB Off Ramp & Sprinkle Road	47	0.491	29,997.12	340,105.77	370,102.89
US-41/M-28 & Front Street	166	1.666	105,349.28	1,194,444.50	1,299,793.78
US-10 BR/ M-20 & Patrick Road	4	0.134	2,517.75	28,546.13	31,063.88
I-75 NB off ramp & M-81/Washington Road	25	0.457	15,733.09	178,380.91	194,114.00
M-53 NB off ramp & 26-Mile	293	1.703	185,440.70	2,102,516.77	2,287,957.47
M-11 & Remembrance Road	177	3.291	111,825.88	1,267,875.88	1,379,701.76
Maple Rd & Farmington Rd	474	4.799	299,818.08	3,399,321.50	3,699,139.58
Total	1,553	17.670	982,413.27	11,138,549.59	12,120,962.86

5.3.1.2 Fuel Costs

The environmental analysis presented earlier showed that converting the 15 intersections to roundabouts would result in annual savings of $2.30 per vehicle. The traffic volume at each of the sites is then used to determine annual fuel savings as shown in Table 39.

Table 39 Annual Fuel Savings by Site

Roundabout Site	AADT	Annual Fuel Savings per Vehicle	Annual Fuel Savings
US-23 NB Off Ramp & Geddes Road	23,388	$ 2.30	$53,705.65
US-23 SB Off Ramp & Geddes Road	22,848	$ 2.30	$52,465.65
US-23 SB & Lee Rd	22,185	$ 2.30	$50,943.21
M-52 & Werkner Road	16,902	$ 2.30	$38,811.90
US-127 BR & Mission Rd	9,647	$ 2.30	$22,152.32
M-5 & Pontiac Trail	54,861	$ 2.30	$125,976.80
NB I-75 Off Ramp & Bristol Road	26,779	$ 2.30	$61,492.37
I-94 EB Off Ramp & Sprinkle Road	40,367	$ 2.30	$92,694.37
I-94 WB Off Ramp & Sprinkle Road	35,253	$ 2.30	$80,951.14
US-41/M-28 & Front Street	36,476	$ 2.30	$83,759.50
US-10 BR/ M-20 & Patrick Road	10,849	$ 2.30	$24,912.46
I-75 NB off ramp & M-81/Washington Road	19,869	$ 2.30	$45,625.00
M-53 NB off ramp & 26-Mile	62,815	$ 2.30	$144,241.50
M-11 & Remembrance Road	19,593	$ 2.30	$44,991.22
Maple Rd & Farmington Rd	36,033	$ 2.30	$82,742.24
Total			

5.3.1.3 Crash Costs

The annual costs of traffic crashes associated with roundabout conversion can be estimated by considering the change in traffic crashes before and after conversion. The unit cost of crashes was estimated per KABCO severity level based on FHWA's *Crash Costs for Highway Safety Analysis* (Harmon et al., 2018). This guidance document suggests the use of comprehensive costs for use as default crash unit cost values for states performing benefit/cost analyses of traffic crashes. Comprehensive costs consider both the tangible economic costs of motor-vehicle crashes, which include wage and productivity losses, medical expenses, administrative expenses, motor vehicle damage, and employers' uninsured costs, in addition to a measure of the intangible costs, including the value of lost quality of life, physical pain, and emotional suffering of people injured in crashes and their families. Thus, the comprehensive costs are much greater than the economic costs alone due to inclusion of the intangible costs. The document provides comprehensive costs per KABCO crash in 2016 dollars, which were converted to 2020 dollars by utilizing the ratio of the 2020 to 2016 annual consumer price index for all urban consumers (CPI-U) (United States Bureau of Labor Statistics, 2022).

The change in traffic crashes after roundabout conversion were estimated using two different methods, which included:

- Raw crash counts
- Empirical Bayes estimates

5.3.1.3.1 Raw Crash Counts

As a first step, raw crash counts were utilized to determine the costs associated with the conversion of an intersection to roundabout. Table 40 shows the aggregated dollar value associated with the annual change in raw crash counts. Site-by-site crash costs before and after the conversion to a roundabout are provided in Appendix-C. The appendix presents detailed calculations on crash costs estimation and benefit-cost ratio estimation on a site-by-site basis.

Table 40 Estimated Annual Crash Frequencies and Associated Costs for Roundabout Conversion based on Raw Crash Counts

Severity	Annual Average Crash Frequency Pre-Construction	Annual Average Crash Frequency Post-Construction	Change in Crash Frequency	Unit Cost per Crash	Change in Annual Crash Cost
K	0.013	0.000	-0.013	$12,176,441.20	$(152,205.52)
A	0.109	0.124	0.016	$706,090.00	$11,068.17
B	0.486	0.375	-0.111	$213,983.00	$(23,668.48)
C	1.968	1.639	-0.329	$135,396.80	$(44,568.04)
PDO	10.361	25.792	15.432	$12,828.20	$197,959.71
Total	12.935	27.930	14.995		$(11,414.16)

5.3.1.3.2 Empirical Bayes Crash Estimates

The crash costs were also calculated for each severity on KABCO scale based on the respective empirical Bayes (EB) index of effectiveness noted in Table 27. These values were then utilized towards determination of the changes in crash costs. The annual aggregated crash estimates before and after roundabout conversion are presented in Table 41. Overall, the results showed that the crash costs increased post roundabout construction which was largely due to a greater number of PDO crashes after roundabout conversion. Crash costs associated with fatal and injury rashes showed significant reductions. Site-by-site crash costs before and after conversion to roundabout based on EB estimates are provided in Appendix-C.

Table 41 Estimated Annual Crash Frequencies and Associated Costs for Roundabout Conversion based on EB Estimates

Severity	Annual Average Crash Frequency Pre-Construction	EB Percent Change in Crash Frequency Post-Construction	EB Change in Crash Frequency	Unit Cost per Crash	Change in Annual Crash Cost
K	0.013	-8%	-0.001	$12,176,441.20	$(12,176.44)
A	0.109	-8%	-0.009	$706,090.00	$(6,133.20)
B	0.486	-8%	-0.039	$213,983.00	$(8,317.08)
C	1.968	-8%	-0.157	$135,396.80	$(21,313.27)
PDO	10.361	110%	11.397	12,828.20	$146,202.73
Total	12.935		14.793		$98,262.73

5.3.2 Agency Costs

The agency costs associated with roundabout conversion include construction costs, utility and signage costs, costs associated with acquiring right-of-way, if needed, and maintenance costs. These costs vary from site to site depending upon site conditions, roundabout size, and whether right-of-way acquisitions are needed. Annual maintenance costs were not available and, as such, are assumed to be equal between roundabouts and traditional intersections. For the purposes of this analyses, an average cost of construction was assumed based on discussion with MDOT staff. These costs were $2,000,000, $2,500,000, and $3,000,000 for single lane, double lane and triple lane roundabouts, respectively. These costs were converted to equivalent annualized costs by assuming a discount rate of 4% and service life of roundabout as 30 years. Table 42 shows the site-by-site equivalent annualized construction costs. These costs were $115,660.20, $144,575.25, and $173,490.30 for single lane, double lane, and triple lane roundabout, respectively.

Table 42 MDOT Roundabout Construction Costs

Site	Type	Construction Costs	Service Life (years)	Discount Rate	Equivalent Annualized Cost
US-23 NB Off Ramp & Geddes Road	Double lane	$2,500,000	30	0.04	$144,575.25
US-23 SB Off Ramp & Geddes Road	Double lane	$2,500,000	30	0.04	$144,575.25
US-23 SB & Lee Rd	Triple lane	$3,000,000	30	0.04	$173,490.30
M-52 & Werkner Road	Single lane	$2,000,000	30	0.04	$115,660.20
US-127 BR & Mission Rd	Single lane	$2,000,000	30	0.04	$115,660.20
M-5 & Pontiac Trail	Triple lane	$3,000,000	30	0.04	$173,490.30
NB I-75 Off Ramp & Bristol Road	Double lane	$2,500,000	30	0.04	$144,575.25
I-94 EB Off Ramp & Sprinkle Road	Double lane	$2,500,000	30	0.04	$144,575.25
I-94 WB Off Ramp & Sprinkle Road	Double lane	$2,500,000	30	0.04	$144,575.25
US-41/M-28 & Front Street	Double lane	$2,500,000	30	0.04	$144,575.25
US-10 BR/ M-20 & Patrick Road	Single lane	$2,000,000	30	0.04	$115,660.20
I-75 NB off ramp & M-81/Washington Road	Single lane	$2,000,000	30	0.04	$115,660.20
M-53 NB off ramp & 26-Mile	Double lane	$2,500,000	30	0.04	$144,575.25
M-11 & Remembrance Road	Double lane	$2,500,000	30	0.04	$144,575.25
Maple Rd & Farmington Rd	Triple lane	$3,000,000	30	0.04	$173,490.30

5.3.3 Benefit-Cost Ratio

Using the estimated costs presented above, benefit/cost ratios were computed separately for each site as well as considering all sites together using the formula below:

$$\frac{B}{C} = \frac{Delay\ savings + Fuel\ savings - Crash\ costs}{Construction\ costs} \qquad \text{Equation 38}$$

Table 43 presents the benefit/cost ratios for roundabout conversion for each of the 15 roundabout sites based on crash estimation method. Detailed methodology for estimation of benefit/cost ratios is provided in Appendix-C. The results showed that the benefit/cost ratios were generally positive indicating that the benefits of roundabout conversion exceeded the disbenefits. Also, the benefit/cost ratios were significantly larger than 1 indicating that the construction costs were generally lower than the benefits. Lastly, the benefit/cost ratios obtained from the two crash estimation methods were similar for most of the sites.

Table 43. Benefit/Cost Ratios for Roundabout Conversion

Roundabout	Type	Benefit/Cost Ratio using Raw Crash Counts	Benefit/Cost Ratio using EB Estimated Crashes
US-23 NB Off Ramp & Geddes Road	Double lane	-2.38	-2.34
US-23 SB Off Ramp & Geddes Road	Double lane	-3.00	-3.26
US-23 SB & Lee Rd	Triple lane	6.52	6.41
M-52 & Werkner Road	Single lane	6.26	3.09
US-127 BR & Mission Rd	Single lane	-0.76	1.43
M-5 & Pontiac Trail	Triple lane	-5.89	1.06
NB I-75 Off Ramp & Bristol Road	Double lane	11.77	8.18
I-94 EB Off Ramp & Sprinkle Road	Double lane	-4.64	3.42
I-94 WB Off Ramp & Sprinkle Road	Double lane	3.93	3.56
US-41/M-28 & Front Street	Double lane	10.22	9.89
US-10 BR/ M-20 & Patrick Road	Single lane	3.12	0.85
I-75 NB off ramp & M-81/Washington Road	Single lane	3.15	1.43
M-53 NB off ramp & 26-Mile	Double lane	35.00	21.45
M-11 & Remembrance Road	Double lane	7.98	10.41
Maple Rd & Farmington Rd	Triple lane	18.62	18.10
Total		6.17	5.91

5.3.4 Benefit-Cost Curves

The annual delay and fuel reductions calculated in the Operational Analyses section (1.2939 hours/vehicle/year and 0.612344 gallons/vehicle/year respectively) as shown in Table 37 were compared with the user delay costs provided by MDOT and the average fuel price in the third quarter of calendar year 2022 to arrive at annual savings per vehicle of $66.80 in delay reductions and $2.30 in fuel savings (a total of $69.09 per vehicle per year). When considering the crash costs, the above analyses showed average crash cost savings of $11,414.16 based on the raw counts. Crash costs actually increased by $98,262.73 annually when considering the EB estimates. These translate to $0.39 and $(3.37) in crash cost savings per vehicle per year when considering raw crash counts and EB estimates, respectively.

Using the *Operations Benefit Cost Spreadsheet* (v3.1) from MDOT's Congestion and Reliability Unit, a one-year and 30-year Benefit-Cost Ratio (BCR) can be calculated. For this analysis, the user delay costs (savings, in this case) and fuel savings have been used on the benefits side while the construction costs provided by MDOT have been used for the costs. Construction costs were converted to equivalent annualized costs by considering 30-year service life and discount rate of 4%. A separate analysis was also conducted by considering annual crash cost savings based on EB estimates. However, the savings from crash reduction benefits were significantly lower than the benefits from delay and fuel savings. As such, benefit/cost ratio were relatively similar whether the crash costs benefits were considered or not. The results for 30-year benefit/cost ratio based on user delay and fuel costs are shown in Table 44. When considering crash costs based on EB estimates, the resulting 30-year benefit/cost ratios are shown in Table 45.

Table 44. 30-Year Benefit/Cost Ratio (User Delay and Fuel Costs Only)

AADT	Annual Benefits (Delay and Fuel)	Construction Costs				
		$1,000,000	$2,000,000	$2,500,000	$3,000,000	$4,000,000
		Equivalent Annualized Costs				
		$57,830.10	$115,660.20	$144,575.25	$173,490.30	$231,320.40
		30-Year B/C Ratio				
5,000	$345,459.73	5.97	2.99	2.39	1.99	1.49
10,000	$690,919.45	11.95	5.97	4.78	3.98	2.99
20,000	$1,381,838.91	23.89	11.95	9.56	7.96	5.97
30,000	$2,072,758.36	35.84	17.92	14.34	11.95	8.96
40,000	$2,763,677.81	47.79	23.89	19.12	15.93	11.95
50,000	$3,454,597.27	59.74	29.87	23.89	19.91	14.93
60,000	$4,145,516.72	71.68	35.84	28.67	23.89	17.92
70,000	$4,836,436.17	83.63	41.82	33.45	27.88	20.91

Ultimately, the threshold at which the construction costs for a roundabout outweigh these operational benefits are rather low; for a single-lane roundabout with an estimated $2 million construction cost, the daily traffic required to break even after 30 years is 1,674 vehicles to 1,735 vehicles (depending upon whether crash costs are considered or not). At this low ADT value, a traditional intersection is unlikely to have an operational need for conversion to a roundabout as little delay would typically be present. Conversely, this also indicates that for new intersections with an expected ADT greater than approximately 1,735 vehicles, a roundabout should generally result in user delay and fuel savings compared to other intersection types. Graphically, the benefit/cost ratio can be estimated for various AADT values and construction costs as shown Figure 23. The curves have been developed considering the EB estimated crash costs along with savings in delay and fuel. The figure clearly shows increasing cost/benefit ratio with traffic volume. This suggests high volume intersection may prove to be good candidates during initial screening process to identify sites to convert to roundabouts.

Table 45 30-Year Benefit/Cost Ratio (User Delay, Fuel Costs, and EB Estimated Crash Costs)

AADT	**Annual Benefits (Delay, Fuel, and EB Estimated Crashes)**	**Construction Costs**				
		$1,000,000	$2,000,000	$2,500,000	$3,000,000	$4,000,000
		Equivalent Annualized Costs				
		$57,830.10	$115,660.20	$144,575.25	$173,490.30	$231,320.40
		30-Year B/C Ratio				
5,000	$333,309.73	5.68	2.84	2.27	1.89	1.42
10,000	$666,619.45	11.36	5.68	4.55	3.79	2.84
20,000	$1,333,238.91	22.73	11.36	9.09	7.58	5.68
30,000	$1,999,858.36	34.09	17.05	13.64	11.36	8.52
40,000	$2,666,477.81	45.46	22.73	18.18	15.15	11.36
50,000	$3,333,097.27	56.82	28.41	22.73	18.94	14.21
60,000	$3,999,716.72	68.19	34.09	27.28	22.73	17.05
70,000	$4,666,336.17	79.55	39.78	31.82	26.52	19.89

When looking at the payback period, the average number of years needed for the operational savings to equal the construction costs may also be calculated for converting a stop-controlled or signalized intersection to a roundabout, as shown in Table 46. The time of return (TOR) in years have been calculated by considering annual benefits that include delay savings, fuel savings, and crash savings (disbenefit) based on EB estimates. The results show that the TOR is significantly less than the service life of a roundabout even at very conservative construction costs. For single and double lane roundabouts with AADT more than 10,000 veh/day, the TOR is generally less than 3 years which means the costs of construction will be covered by user benefits in less than 3 years of roundabout construction.

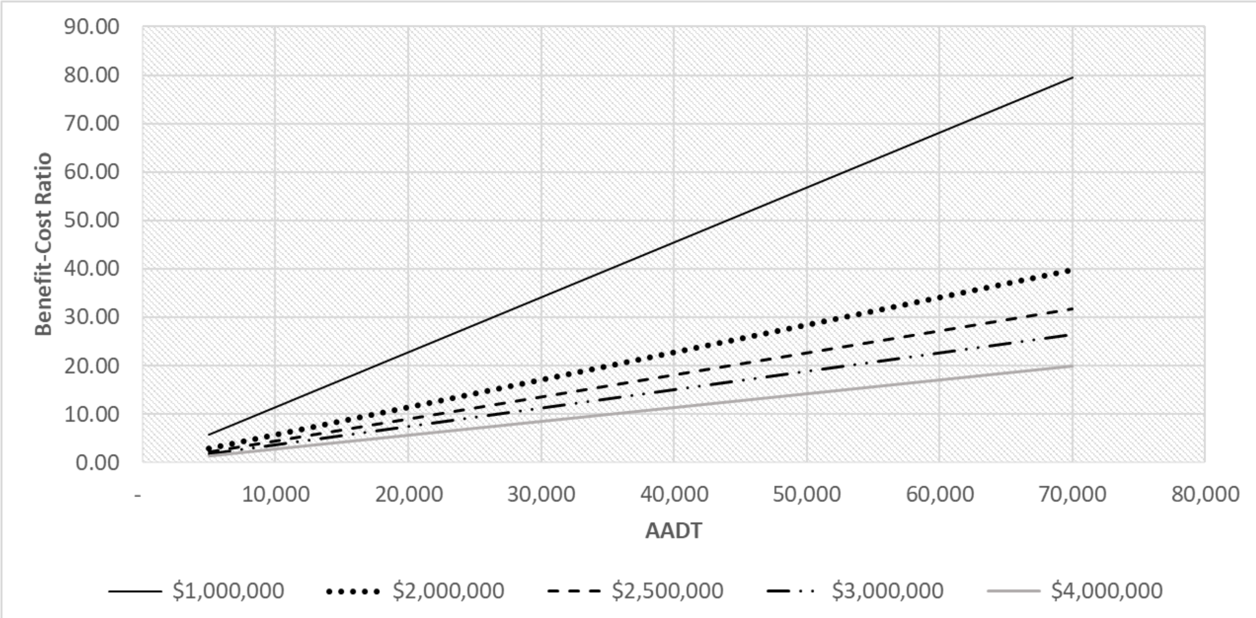

Figure 23 Benefit-Cost Curves for Roundabout based on 30-Year Service Life

Table 46 Time of Return (User Delay, Fuel and EB Estimated Crash Savings) for Roundabouts

AADT	Annual Benefits (Delay, Fuel, and EB Estimated Crashes)	Construction Costs				
		$1,000,000	$2,000,000	$2,500,000	$3,000,000	$4,000,000
5,000	$328,609.73	3.04	6.09	7.61	9.13	12.17
10,000	$657,219.45	1.52	3.04	3.80	4.56	6.09
20,000	$1,314,438.91	0.76	1.52	1.90	2.28	3.04
30,000	$1,971,658.36	0.51	1.01	1.27	1.52	2.03
40,000	$2,628,877.81	0.38	0.76	0.95	1.14	1.52
50,000	$3,286,097.27	0.30	0.61	0.76	0.91	1.22
60,000	$3,943,316.72	0.25	0.51	0.63	0.76	1.01
70,000	$4,600,536.17	0.22	0.43	0.54	0.65	0.87

5.4 Summary

The analysis of roundabouts in Michigan has demonstrated significant savings both operationally as well as environmentally. Thirteen of fifteen pairs of study locations indicated reductions in delay of 56.6 percent to 67.3 percent on an hourly basis which on an annual basis average to 1.3 hours per vehicle. Additionally, the reduction in idling demonstrates savings in both fuel and emissions. Benefit/cost analysis also showed that conversion of a traditional intersection to a roundabout will generally result in benefits that significantly outweigh the costs over a longer period of time. While future changes to the vehicle fleet (e.g. electrification) may reduce the future emissions savings, the time savings (and associated reduction in user delay costs) will endure.

6 CONCLUSIONS AND RECOMMENDATIONS

The purpose of this research was to evaluate differences in the operational and safety performance of roundabouts as compared to traditional (i.e., signalized and stop-controlled) intersections. The study also assessed environmental impacts and assessed the cost-effectiveness of roundabouts compared to conventional intersections. The following sub-sections present the conclusions of this study followed by general recommendations.

6.1 Impacts on Driver Behavior

Investigation of driver behavior using field data showed variation in driver behavior based on site characteristics. Driver speed profiles when approaching a roundabout tended to be influenced by vehicle type, traffic volume on the cross road, type of advance roundabout warning sign, and distance from the yield line. However, the magnitude of these effects was more significant and pronounced nearer to the roundabout yield line compared to any location upstream. This is expected since speeds will be affected only near to the roundabout entry.

In terms of gap behavior, drivers tended to be influenced by roundabout geometry and site characteristics such as roadway context. Accepted gaps were smaller on multilane roundabouts compared to single lane roundabouts. Accepted gaps were also smaller on three-legged roundabouts compared to four-legged roundabouts. Similarly, drivers tended to accept smaller gaps at roundabouts located in rural areas compared to urban areas, and on roundabouts on surface roads compared to roundabouts on ramp terminals.

These results also translated to yielding behavior of drivers towards other vehicles in the traffic circle. Triple lane roundabouts exhibited higher yielding rates than double and single lane roundabouts which is due to the complex nature of these roundabouts and generally higher volumes which forces drivers to slow down near the yield line. Yield rates were also significantly lower on roundabouts located on ramp terminals. However, vehicles entering the roundabout through the major road were less likely to yield compared to drivers entering the roundabout through an exit ramp or a minor approach. Additionally, yield rates were lower on rural roundabouts compared to urban roundabouts. Roundabouts located at US-10 at M-30, and US-23 at Geddes Road exhibited the lowest yield rates (close to 81%) among all the 18 sites where the data were collected. Both of these sites are located on interchange.

When yielding to pedestrians, US-23 at Geddes Road and M-53 at 26 Mile Road showed very low yield rates with only 45% and 43%of the drivers yielding to pedestrians, respectively. Again, both of these sites are located on an interchange. Sites with PHB installed for pedestrians exhibited the highest pedestrian yield rates with nearly 90% of the drivers yielding to pedestrians. Roundabouts with multiple approach lanes showed that drivers in the near lane were more likely to yield to pedestrians compared to drivers in the far lane.

6.2 Impacts on Safety

In general, roundabouts were shown to have positive impacts on safety performance. Certain severe crash types (e.g., head-on and right-angle crashes) are eliminated due to the roundabouts' unique geometric designs. In 2011, MDOT investigated the safety performance of the roundabouts in Michigan. It was found that the roundabout improved the safety performance compared to the traditional intersections. With the increased roundabouts in Michigan, this report re-evaluated the safety performance of roundabouts in Michigan and compared the results to the 2011 MDOT report.

A total of 180 roundabouts in Michigan were initially identified from various sources (i.e., WSP, MDOT, Kittelson & Associates, and Michigan Auto Law). Meanwhile, 157 companion control sites were manually selected in such a manner that the traffic volume and geometric characteristics were similar to the roundabouts. Multiple analyses were conducted to understand the safety performance of roundabouts in Michigan comprehensively. The analysis period of this study was from 2004 to 2019. The final study sites varied for different analyses due to the data availability and analysis purposes.

At first, a naïve before and after comparison was performed among 142 roundabouts (i.e., 92 single-lane, 42 double-lane, and 8 triple-lane roundabouts). The results showed that the total crashes increased by 8%, 123%, and 233% on single-, double-, and triple-lane roundabouts, respectively, after the construction of roundabouts. The injury crashes decreased by 34% and 12% on single- and double-lane roundabouts, but it increased by 40% on triple-lane roundabouts.

Although it should be noted that only 8 triple lane roundabouts were investigated in this study which is very small sample size compared to single and double lane roundabouts.

Additionally, the EB analysis was conducted using 97 roundabouts and 92 control sites to evaluate the performance of roundabouts converted from other types of intersections. Multiple SPFs and CMFs were developed based on the number of approaches (i.e., three- and four-leg) and types of previous traffic controls (i.e., signalized and stop-control). The findings showed similar trends with the naïve before and after study. The total crashes increased for all types of roundabouts, while the fatal and injury crashes reduced on single and double-lane roundabouts and increased on triple-lane roundabouts.

Moreover, the analysis also indicated that the total crashes increased by 105% and 127% after converting signalized and stop-control intersections to roundabouts, respectively. Ten percent reduction and 4% increase were observed in fatal and injury crashes for the roundabouts converted from signalized and stop-control intersections, respectively. The cross-sectional analysis also showed similar findings. The conversions of signalized and stop-control intersections to roundabouts increased total crashes by 58% and reduce fatal and injury crashes by 27%, on average, for locations in this study.

In comparison to the results of the prior MDOT study (Bagdade et al., 2011), the results of this research have shown larger increases in total crashes and smaller decreases in injury crashes as compared. It is important to note that there are several potential reasons for this result. First, the number of roundabouts included in this study was significantly larger as compared to the prior study. Related to this point, there is also greater diversity in terms of the characteristics of the roundabouts that have been included, as well as the contextual environments in which they are located. It is also important to note that the sites that were initially converted to roundabouts tended to experience the largest numbers of crashes during the pre-conversion period. As a consequence, the latter conversions tended to experience fewer crashes, both overall and compared to other locations. While some of these differences are accommodated through the EB analysis, it is likely that the inclusion of a larger number of sites, including those with lower crash histories plays a role in these findings.

6.3 Impacts on Delay and Environment

The effect of conversion of conventional intersection to a roundabout on vehicle delay were investigated using Rodel roundabout analysis software. Corresponding changes in level of service (LOS) were also assessed based on delay using Synchro v11. Relevant data were collected for roundabout and their corresponding intersections. Results showed that the average reduction in delay on roundabouts was 57%, 67%, and 66% during the morning peak hour, off peak period, and evening peak hour, respectively. Also, these reductions were more pronounced in rural areas compared to urban areas.

Under certain assumptions, the analyses also showed that 12.76 seconds of delay is saved per vehicle on roundabouts. Corresponding annual savings in fuel consumption amount to $2.30 per vehicle per year. Converting an intersection to a roundabout could also result in saving nearly $66.8 per vehicle per year. On an intersection with a traffic volume of 10,000 vehicles per day, these savings amount to 12,937.2 hours per year saved in reduced delay or $66,800 in annual delay savings, and $23,000 in annual fuel savings. However, it should be noted that these results are based on analyses of only 15 roundabout-intersection pairs which is a small sample of all roundabouts in the state of Michigan.

From an economic standpoint, analysis also showed that the benefits associated with a roundabout conversion are generally more than the costs associated resulting in a favorable long-term investment for roadway agencies. Converting a traditional stop-controlled or signalized intersection to a roundabout usually resulted in a benefit/cost ratio of 5.9. Annual user benefits in terms of reduced delay and fuel savings amounted to nearly $69.09 per vehicle per year. Crash cost savings were relatively small compared to environmental savings and amounted to $0.39 and $(3.37) savings per vehicle per year when considering crash estimates based on observed raw crash counts and EB estimated crash counts, respectively.

6.4 Conclusions and Recommendations

- Converting conventional intersections to roundabouts was shown to significantly increase total crashes and significantly reduce fatal and injury crashes. With that being said, intersections that experience a larger proportion of fatal and injury crashes present the best candidate locations for conversion to roundabouts.

- In general, the crash cost savings from the reduction in more severe crashes tend to significantly outweigh the increase in lower-cost property-damage-only crashes. However, these differences should be considered on a case-by-case basis. Converting intersections to roundabouts consistently showed significant savings in terms of delay reductions and fuel savings. Ultimately, cost-benefit analyses play an important role in deciding whether to convert an intersection to a roundabout or not.

- Although the sample size was small, roundabouts with three circulating lanes experienced increases in both total and fatal and injury crashes after conversion. Consequently, it is recommended that conversions be limited to two circulating lanes, wherever possible. Due to their complexity, signage, pavement markings, and the provision of advance information to aid in navigation are particularly important where triple-lane roundabouts are constructed. These same issues are ultimately important for all roundabouts, though it should be noted that the concerns with respect to triple lane roundabouts are noted in the MDOT Roundabout Guidance document (Bott et al., 2007).

- In terms of driver behavior, special consideration may be warranted for roundabouts located at interchanges. In general, these roundabouts tended to exhibit much lower yielding rates compared to other roundabouts, especially when drivers encountered pedestrians. This may be due to higher approach speeds, and driver expectation of the lower likelihood of presence of pedestrian at interchanges. Improving driver visibility towards pedestrians standing at the curb waiting to cross the road may improve yielding behavior of drivers towards pedestrians. Also, measures to reduce approach speeds of vehicles as they are exiting the freeway and approaching the roundabout should be considered. Use of dynamic speed feedback sign (DSFS) can be considered when looking into such measures, including combining DSFS with an advanced roundabout warning sign to inform drivers of their speeds. However, proper field studies need to be conducted to evaluate the effects of such traffic control devices at roundabout approaches. A general recommendation regarding measures to control driver speed at roundabouts on high-speed approaches and roundabouts on interchanges could be included in the Section 5.3 of the MDOT Roundabout Guidance document (Bott et al., 2007).

- The two sites with a PHB showed that nearly 90% of drivers yielded to pedestrians. In comparison, the roundabout located on Geddes at Earhart Road has traditional pedestrian crossing and showed a yield rate of 86% towards pedestrians. As such, installing PHB at roundabouts may not necessarily improve yielding rates by a significant amount to justify the investment. During field data collection, it was noted that the pedestrian and bicycle volumes at nearly all of the roundabouts were very low. Thus, sites in highly urban areas with significant ped-bike volumes may prove to be good candidates to install PHB. It should be noted that Section 4.9 in the current MDOT Roundabout Guidance mentions installing PHB facilities at sites where pedestrian volume is high. The findings from the current study support these existing guidelines. Roadway agencies should only consider installing PHB at urban locations with significant ped-bike activity. Cost-benefit analysis may also be considered to justify PHB installation.
- The results of this research have led to a series of SPFs and CMFs that can be used by MDOT for the purposes of planning- and design-level analyses when considering roundabout construction or conversion projects. Separate CMFs and predictive models have been estimated for various scenarios of interest, providing improved flexibility for designers when comparing alternatives at intersections that are promising locations for roundabout conversions. For example, these tools can be used to estimate the number of crashes by type, and to compare expected safety performance based on general characteristics (e.g., number of approach legs, number of circulating lanes), as well as detailed geometric characteristics, such as the inscribed diameter and entry width.

6.5 Limitations and Future Work

The results from this study were generally consistent with the literature and the prior MDOT study published in 2011. Nevertheless, the present study has some limitations that should be noted.

- **Speed Selection Behavior:** The field data collection included collecting data related to driver speed selection as they approach a roundabout, i.e., speed profiles were collected only till the yield line. The present study did not assess driver behavior within the roundabout circle. Thus, future research can collect driver speed profiles within the roundabout. Additionally, data could also be collected to investigate driver behavior as they exit a roundabout.

- **Yielding Behavior:** Driver's yielding behavior was investigated towards other vehicles in the circle and also towards pedestrians at crossing on roundabout entry. Yielding behavior of drivers towards pedestrians at roundabout exit is an area that needs to be investigated. During field studies it was observed that drivers tend to accelerate quickly as soon as they are on their desired exit.

- **Driver Characteristics:** The driver's speed selection behavior, gap behavior and yielding behavior were investigated based on site characteristics. The analysis can be strengthened by considering driver demographic characteristics in the analyses. Future research can consider collecting data related to driver demographics either at individual driver level or aggregated site level.

- **Safety Analysis:** The number of triple lane roundabouts was very less in the present analysis compared to single and double lane roundabouts. As such, additional triple lane roundabouts may be identified from other states (since all Michigan roundabouts were considered in the present study) and included in the safety analyses to evaluate their safety benefits in greater detail.

- **Delay and Environmental Benefits:** Although the present study showed significant benefits in terms of reducing delay and fuel consumption due to converting a conventional intersection to a roundabout, and consequently a positive benefit/cost ratio greater than 1, the results are based on analyses of only 15 roundabout-intersection pairs which is a small sample of all roundabouts in the state of Michigan. As such, future studies can consider including additional roundabouts in the analysis.

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8 APPENDIX A

This appendix provides the average driver speed selection profile for each of the sites where lidar speed data were collected as a part of field evaluations. The profiles are shown based on speeds interpolated at every 50 ft interval which are averaged across all drivers. The error bars are also shown at every 50 ft increment. The location of the advanced roundabout warning sign is also marked in the figure if the sign was present and speed measurements were recorded at the location of the sign. At some of the sites, the vehicle was parked downstream of the roundabout warning sign due to lack of suitable parking location upstream of the sign. Additionally, for sites with pedestrian crossing, separate average speed profiles are shown based on whether the driver yielded to the pedestrian (marked as 'yield' in the figure) or not (marked as 'not yield' in the figure). If no pedestrian was present at the time the subject vehicle approached the pedestrian crossing, the speed profile was recorded as 'no ped' case.

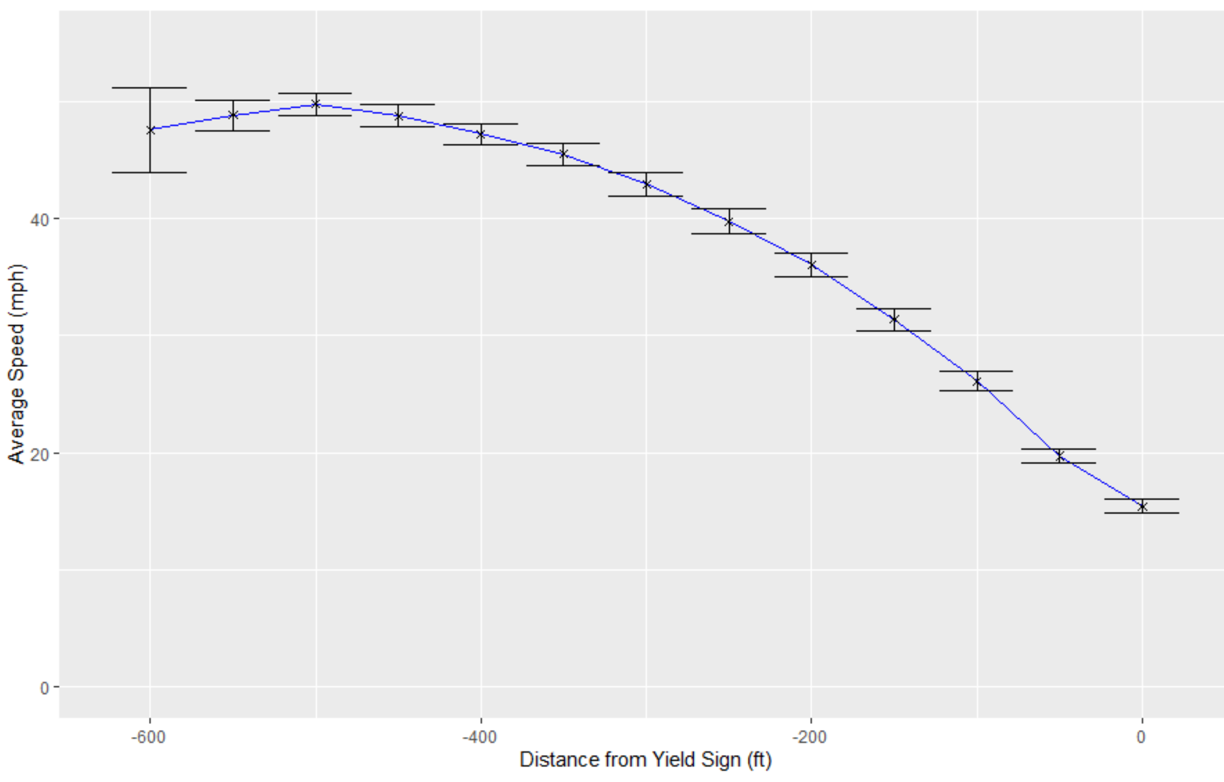

Figure 24 Interpolated Speed Profile at Every 50ft Interval for Site 1 NB Approach (M-5 at Pontiac Trail)

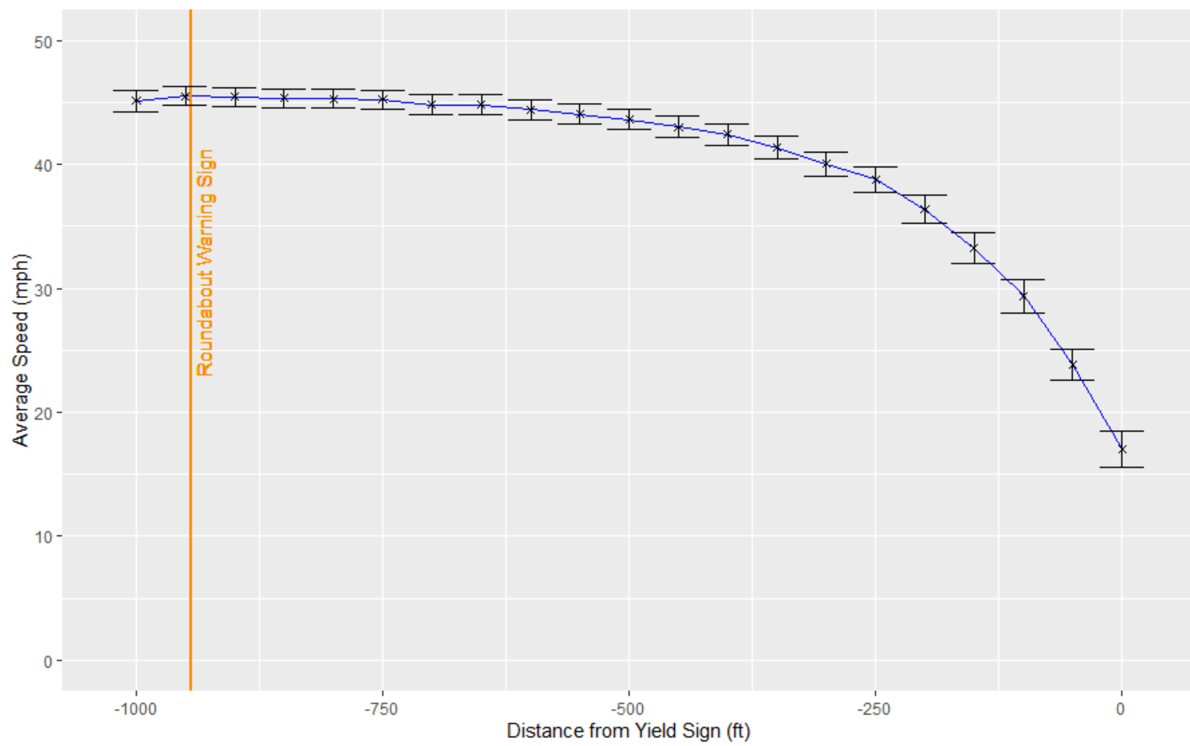

Figure 25 Interpolated Speed Profile at Every 50ft Interval for Site 1 WB Approach (M-5 at Pontiac Trail)

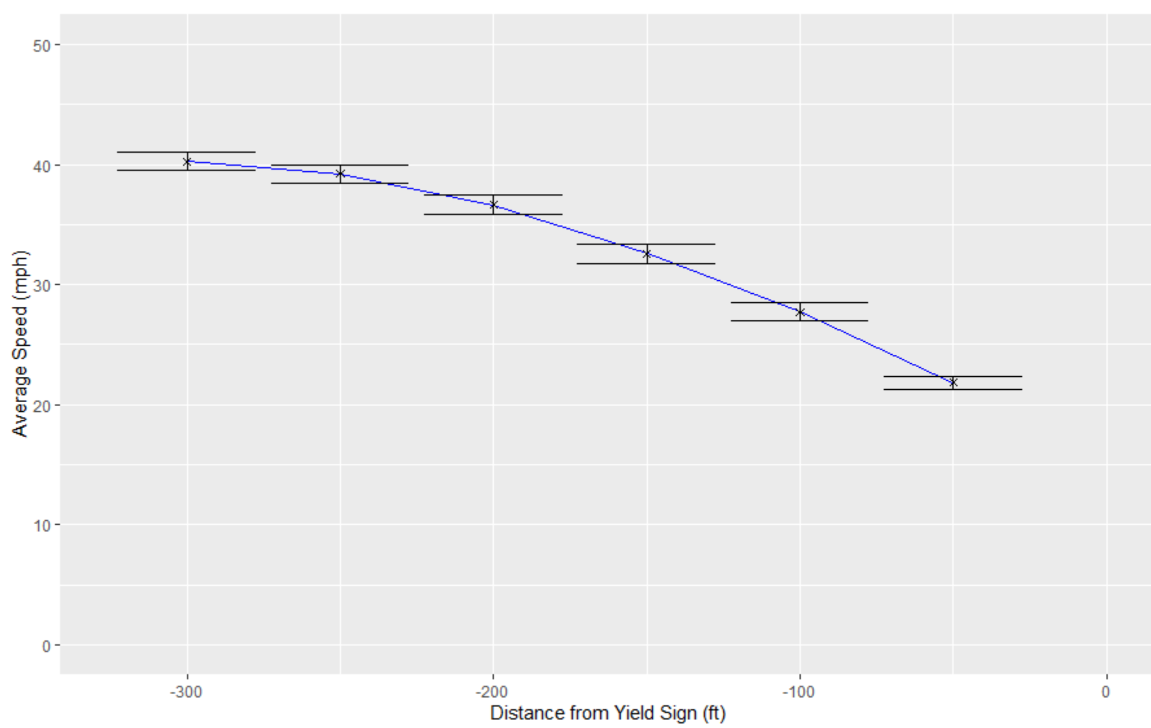

Figure 26 Interpolated Speed Profile at Every 50ft Interval for Site 2a WB Approach (I-75 at Monroe M-46)

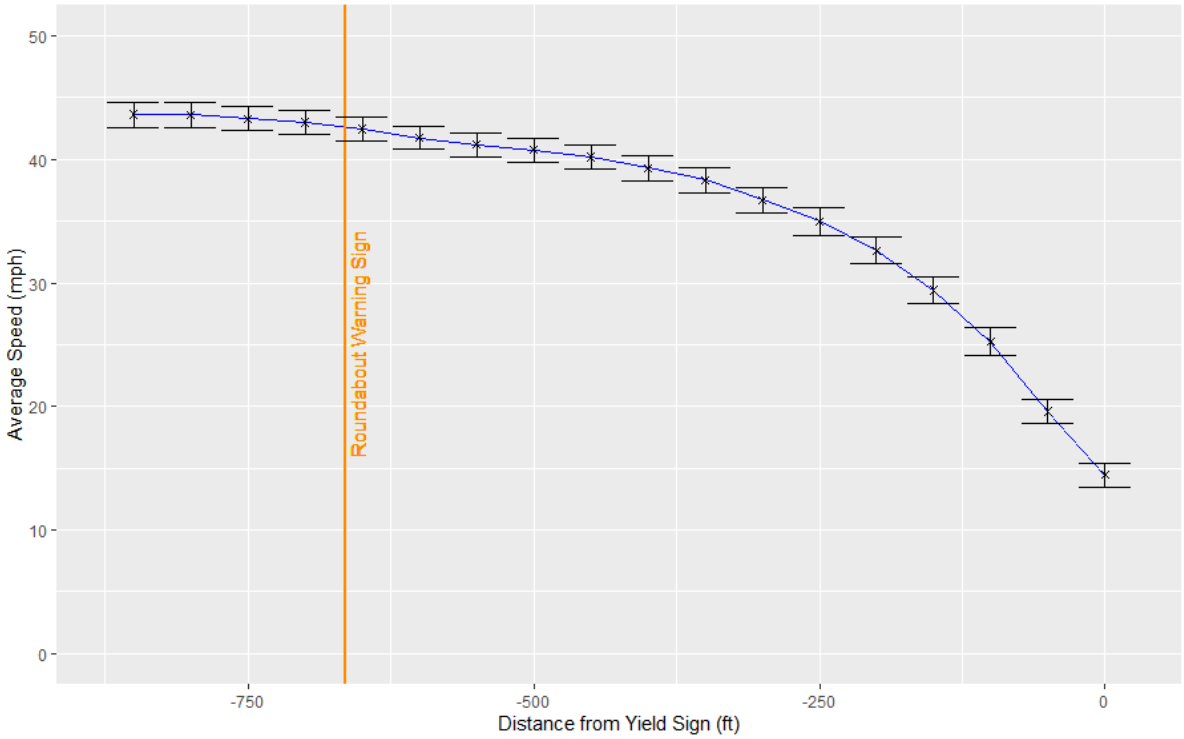

Figure 27 Interpolated Speed Profile at Every 50ft Interval for Site 3a WB Approach (I-75 at M-81)

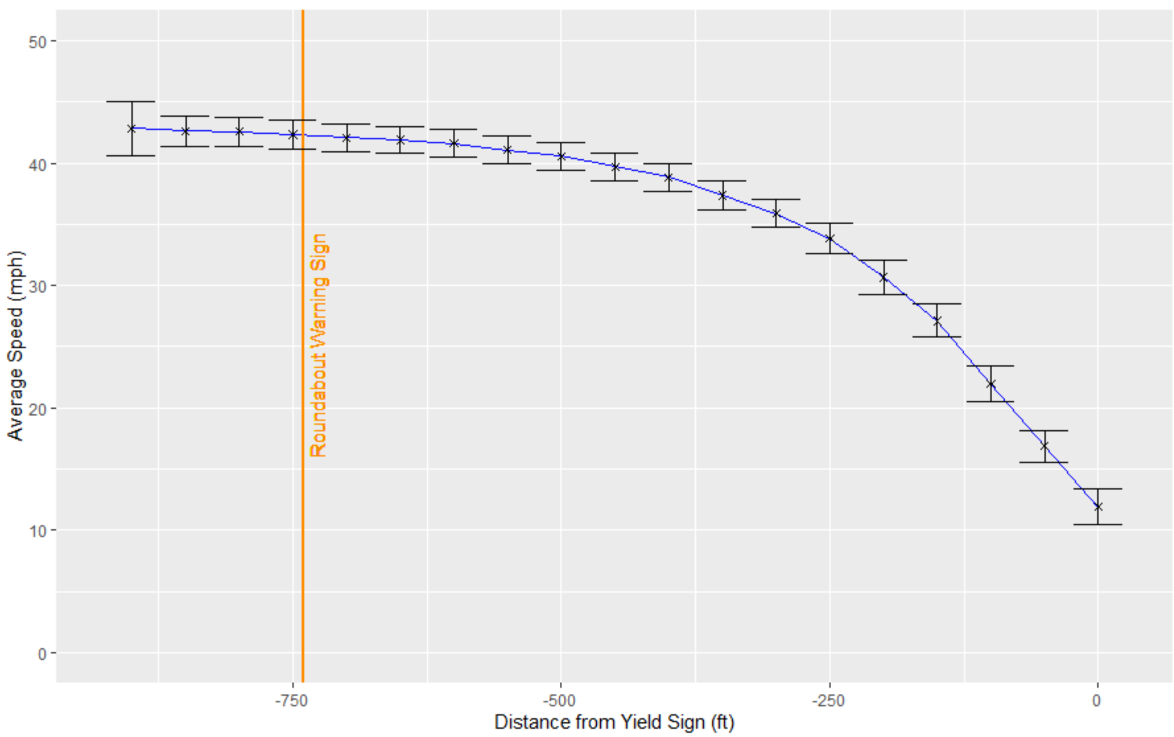

Figure 28 Interpolated Speed Profile at Every 50ft Interval for Site 3b EB Approach (I-75 at M-81)

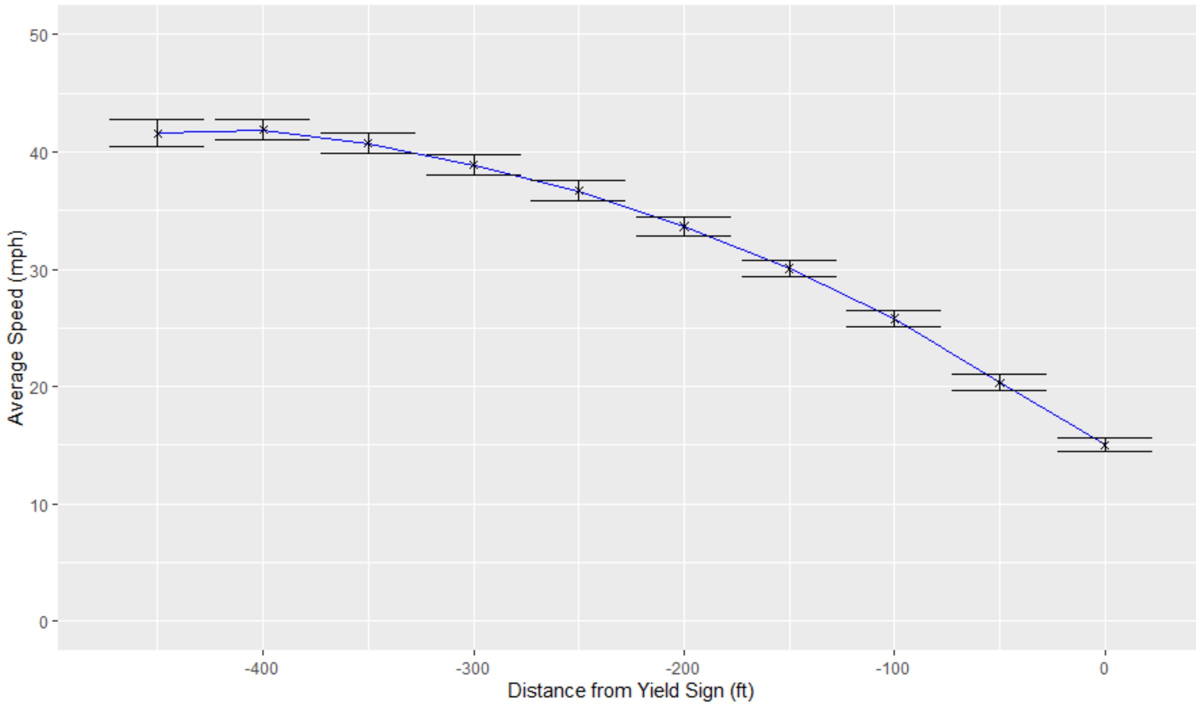

Figure 29 Interpolated Speed Profile at Every 50ft Interval for Site 4 SB Approach (US-10 at M-30)

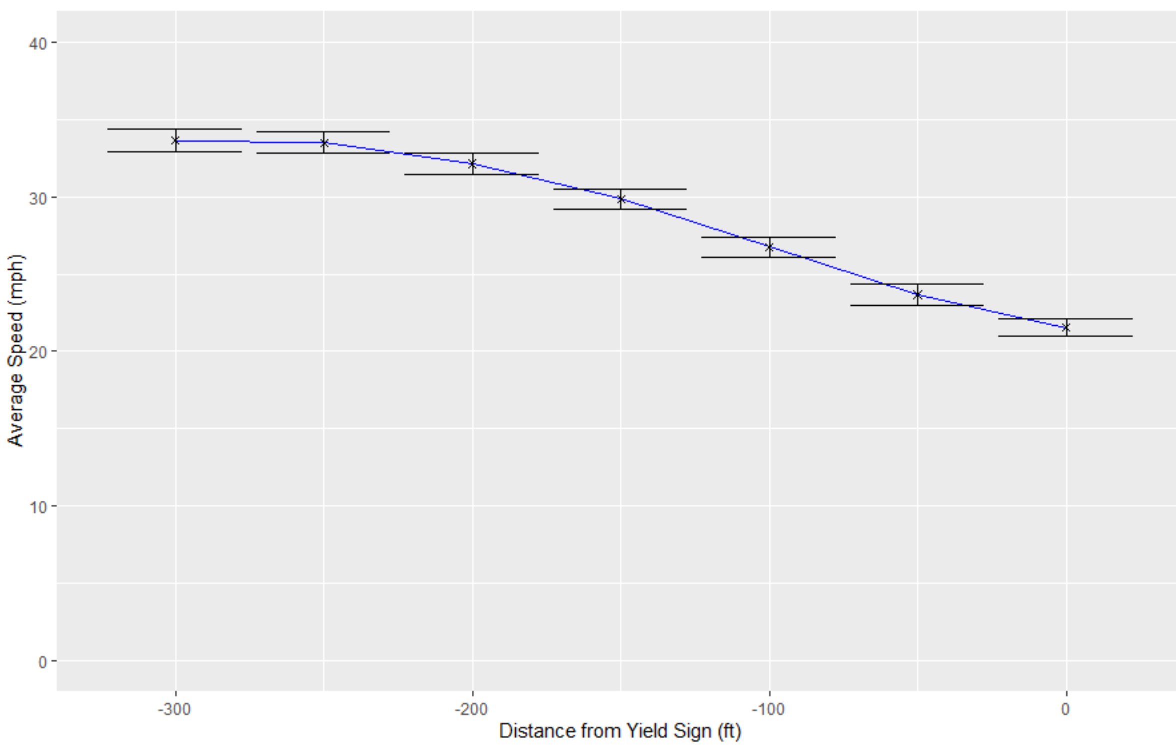

Figure 30 Interpolated Speed Profile at Every 50ft Interval for Site 5 SB Approach (US-127 BR at Mission Road)

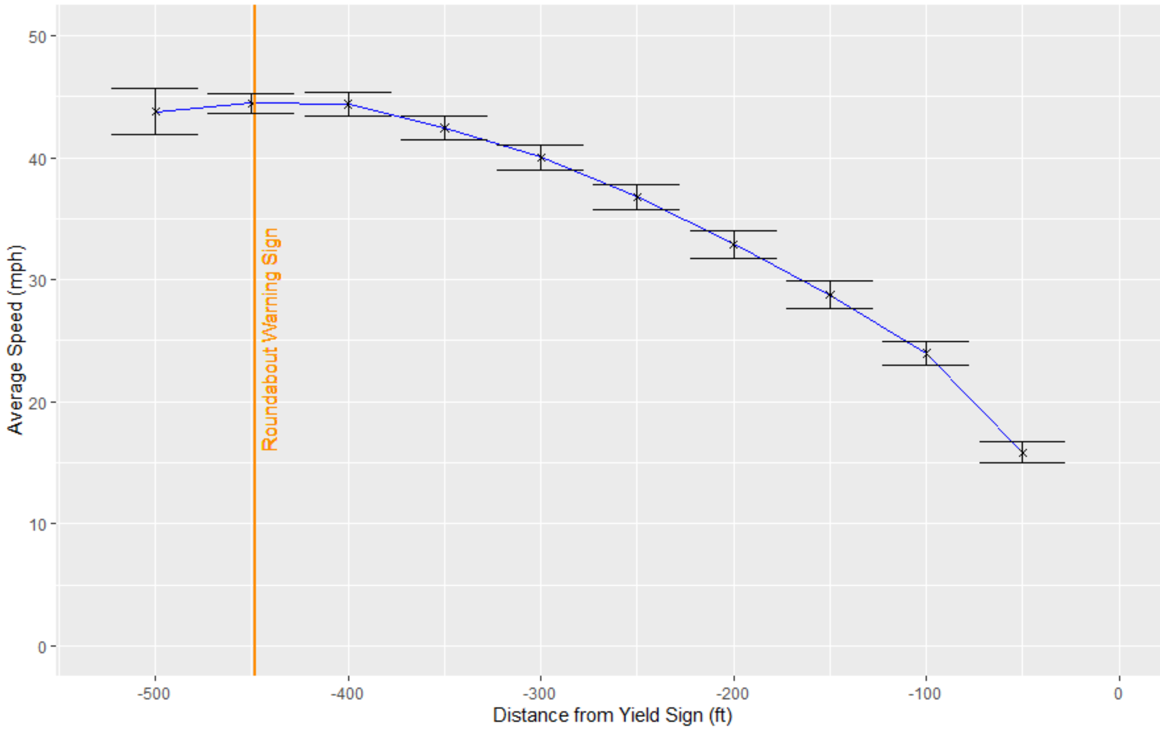

Figure 31 Interpolated Speed Profile at Every 50ft Interval for Site 6 SB Approach (US-23 at Lee Road)

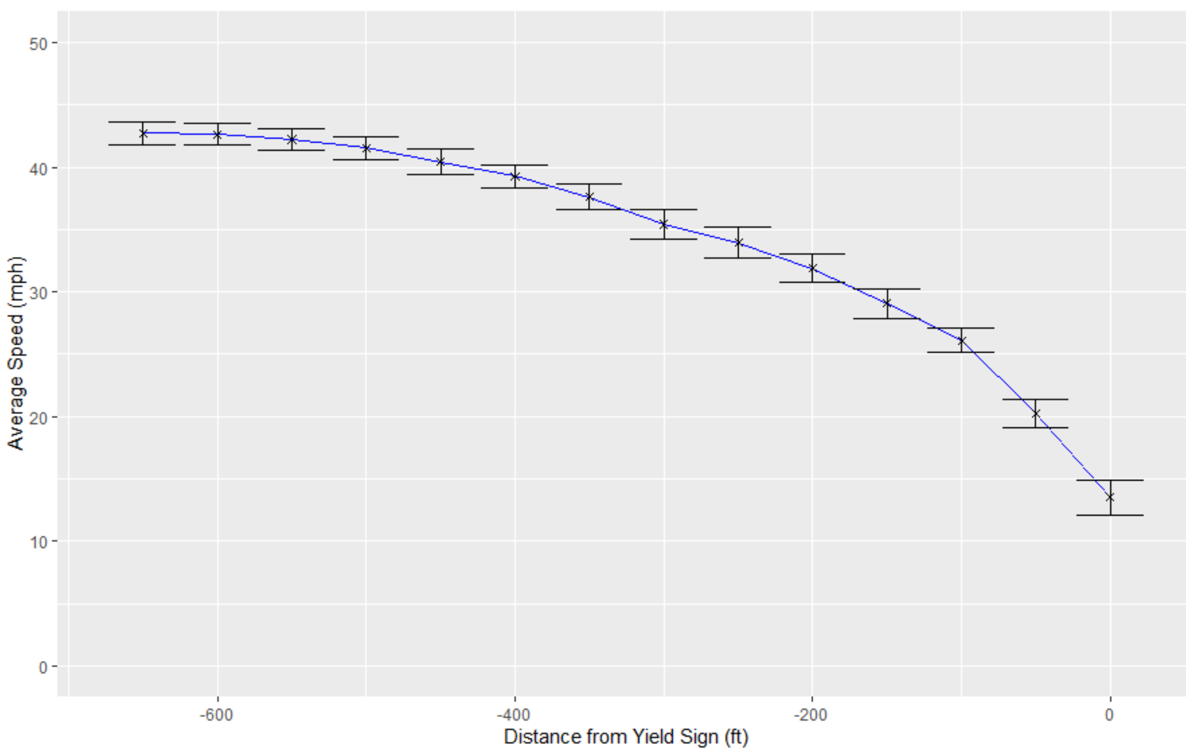

Figure 32 Interpolated Speed Profile at Every 50ft Interval for Site 7 NB Approach (I-94 at Sprinkle Road)

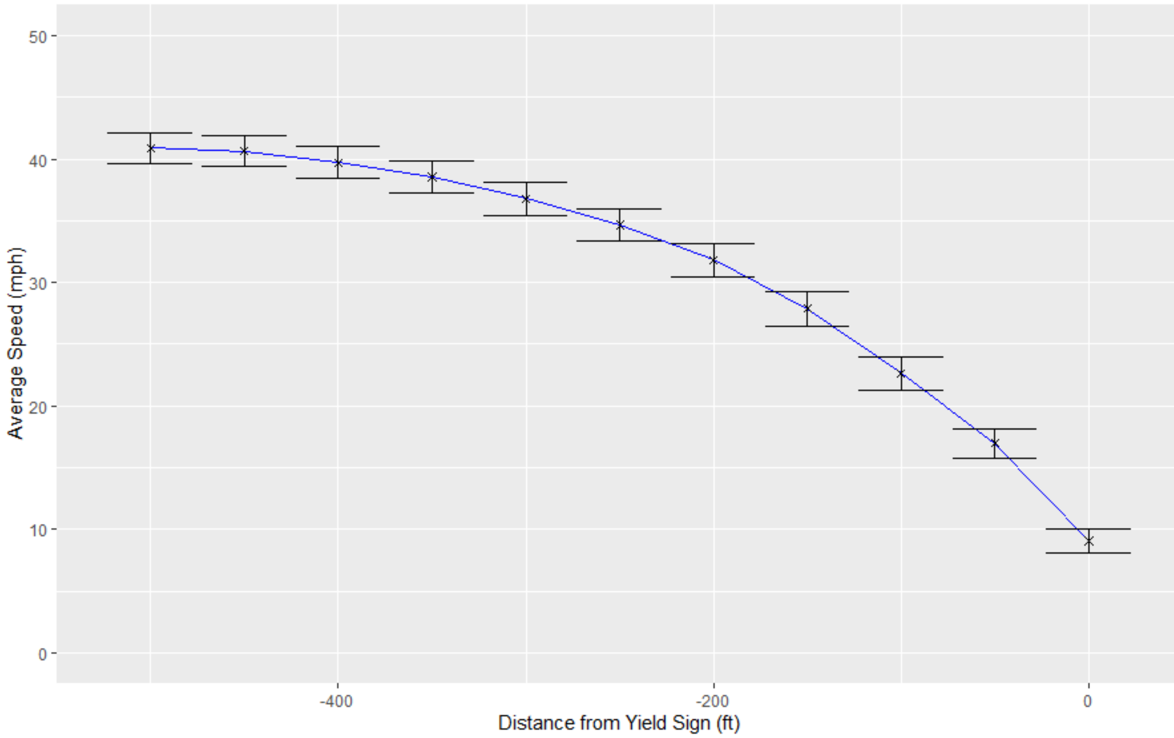

Figure 33 Interpolated Speed Profile at Every 50ft Interval for Site 8 WB Approach (I-94 at Sprinkle Road)

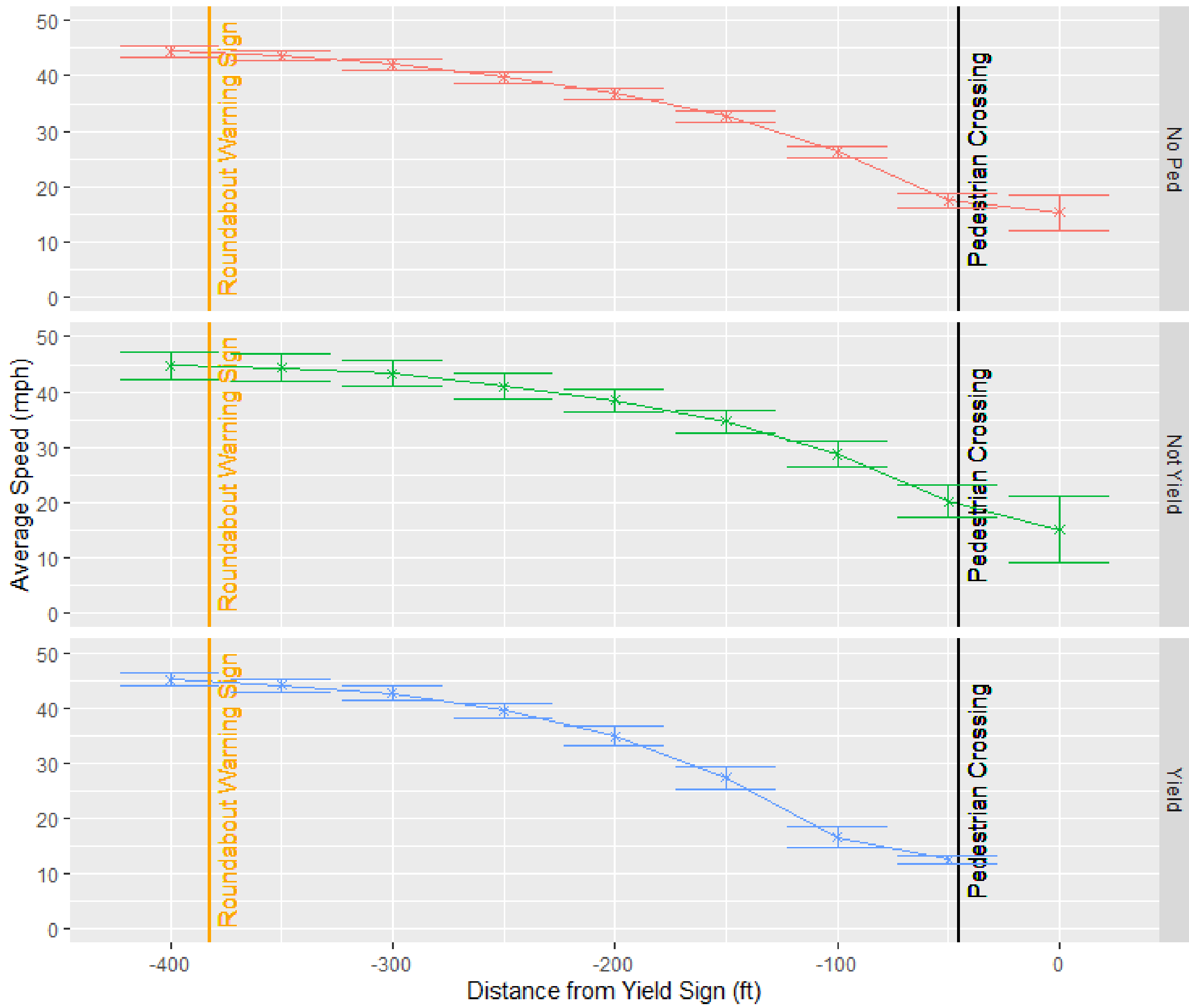

Figure 34 Interpolated Speed Profile at Every 50ft Interval for Site 11 NB Approach (US-23 at Geddes Road)

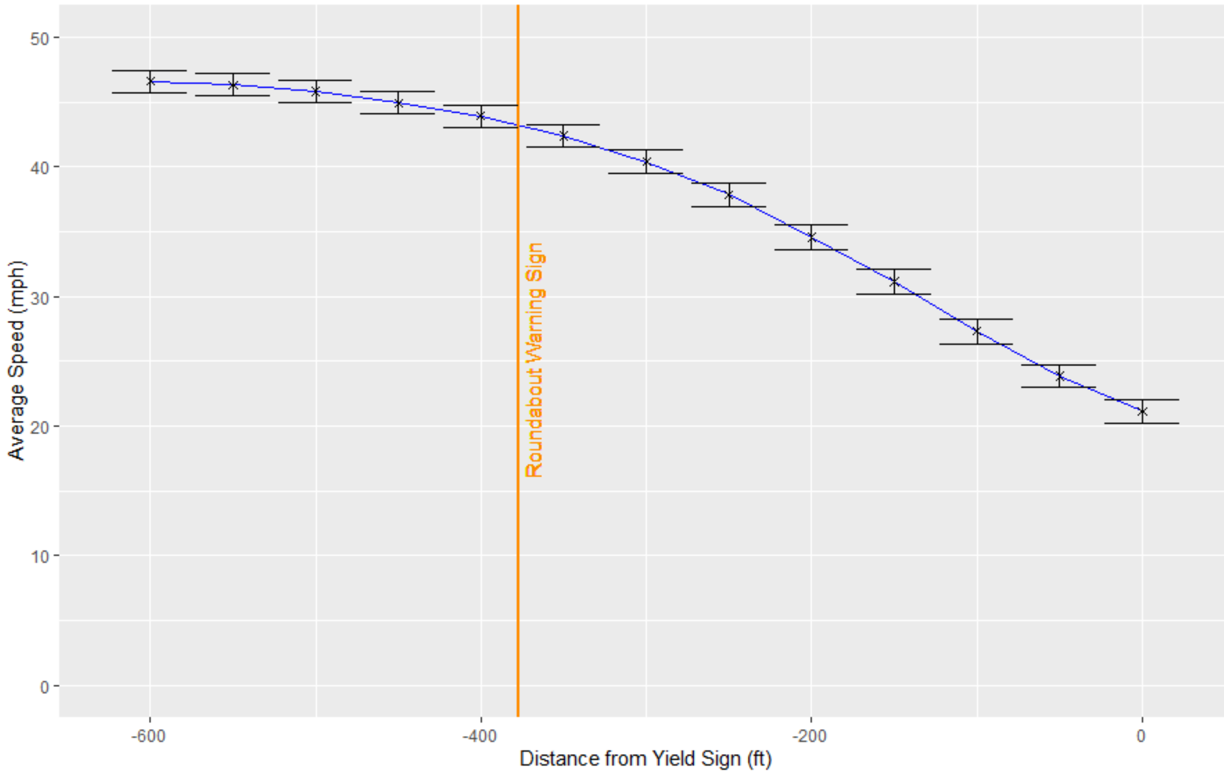

Figure 35 Interpolated Speed Profile at Every 50ft Interval for Site 12 NB Approach (M-52 at Werkner Road)

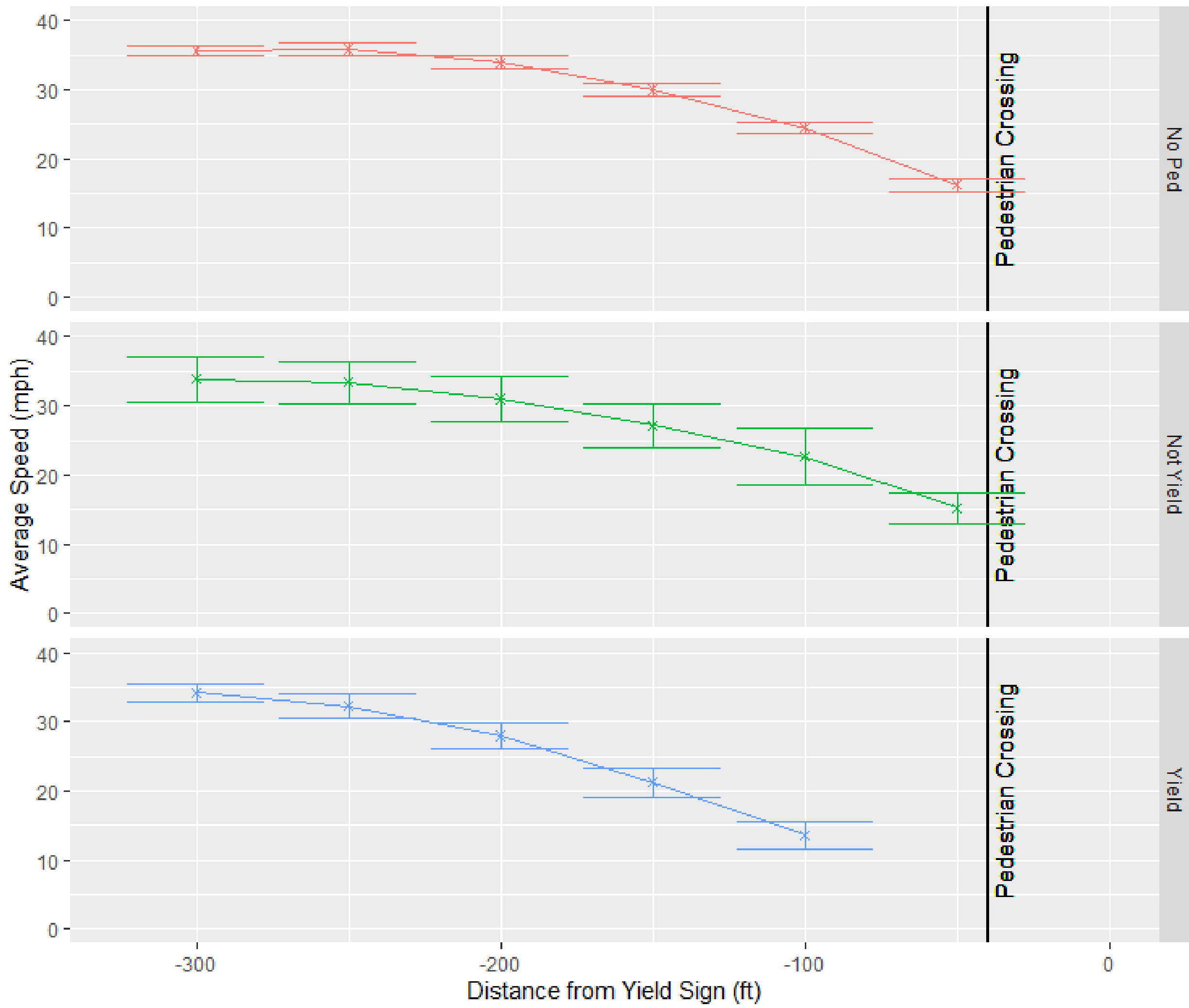

Figure 36 Interpolated Speed Profile at Every 50ft Interval for Site 13 EB Approach (Farmington at Maple Road)

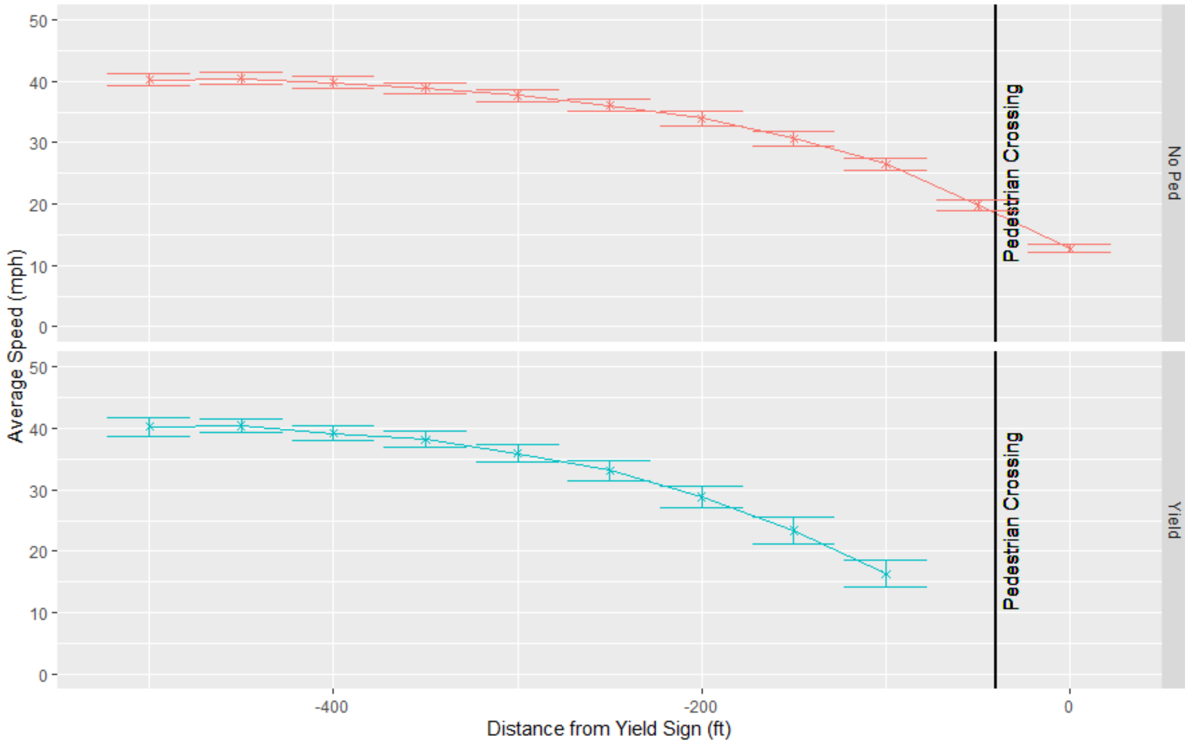

Figure 37 Interpolated Speed Profile at Every 50ft Interval for Site 14 WB Approach (Drake at Maple Road)

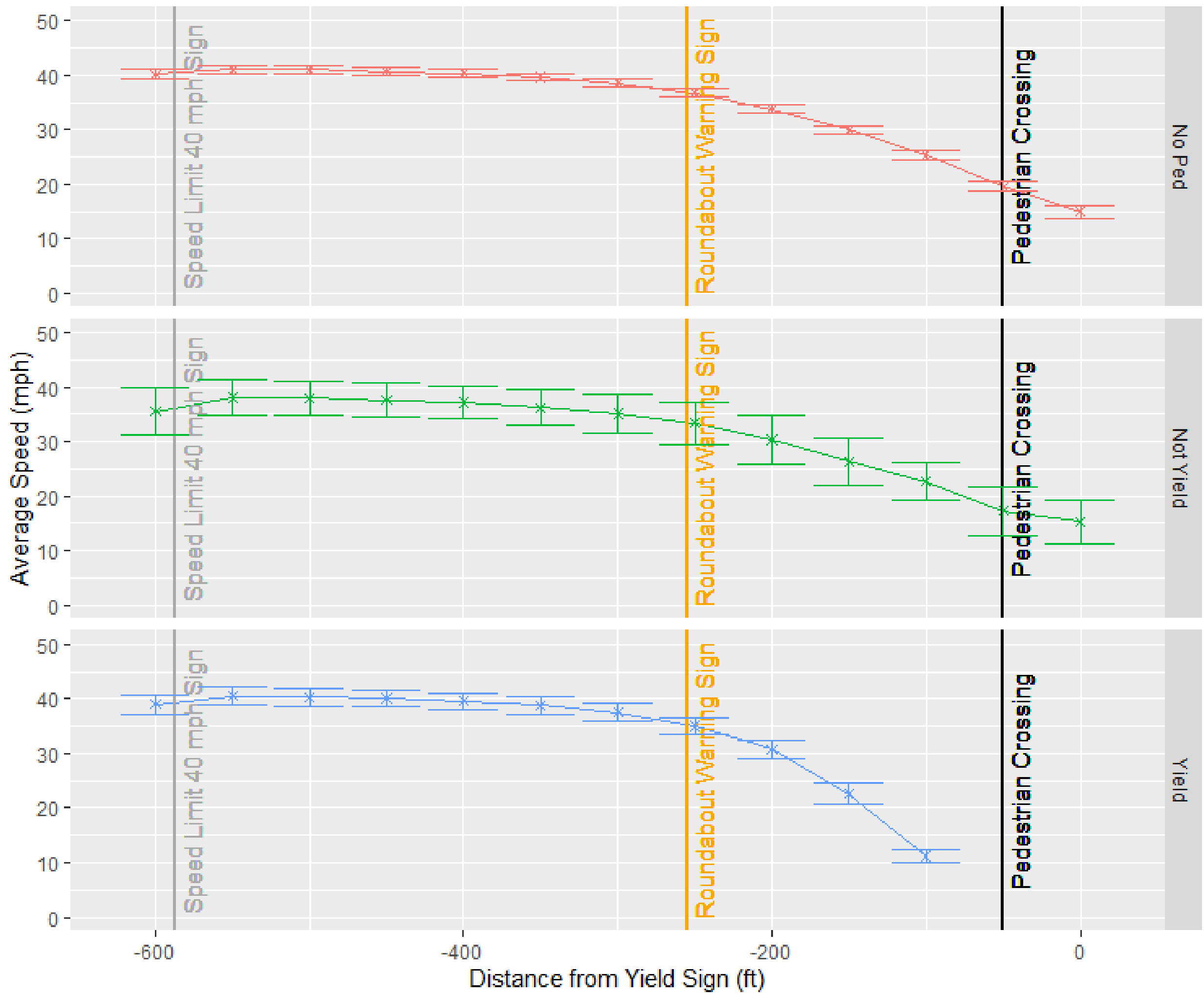

Figure 38 Interpolated Speed Profile at Every 50ft Interval for Site 15 EB Approach (Geddes at Earhart Road)

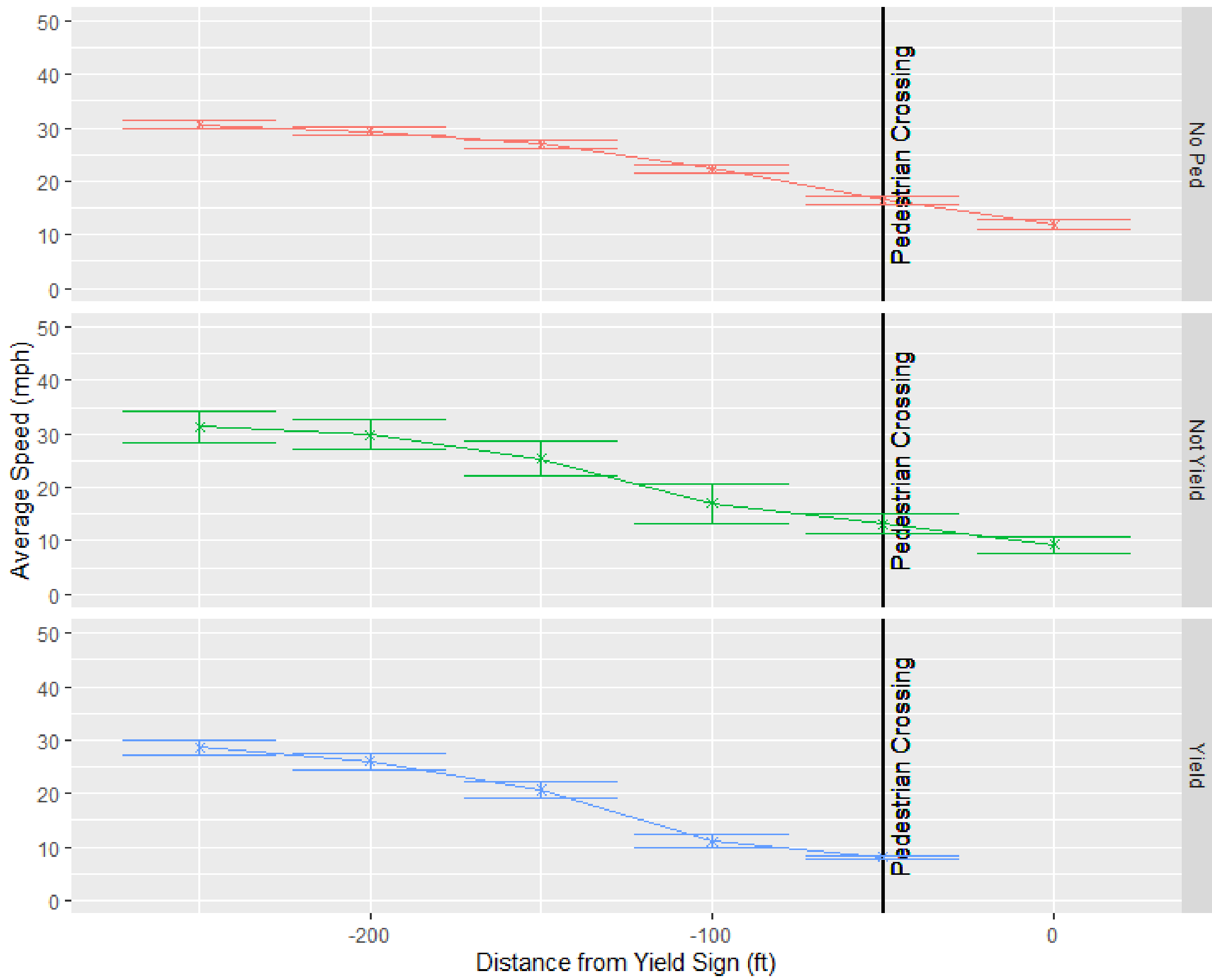

Figure 39 Interpolated Speed Profile at Every 50ft Interval for Site 15 SB Approach (Geddes at Earhart Road)

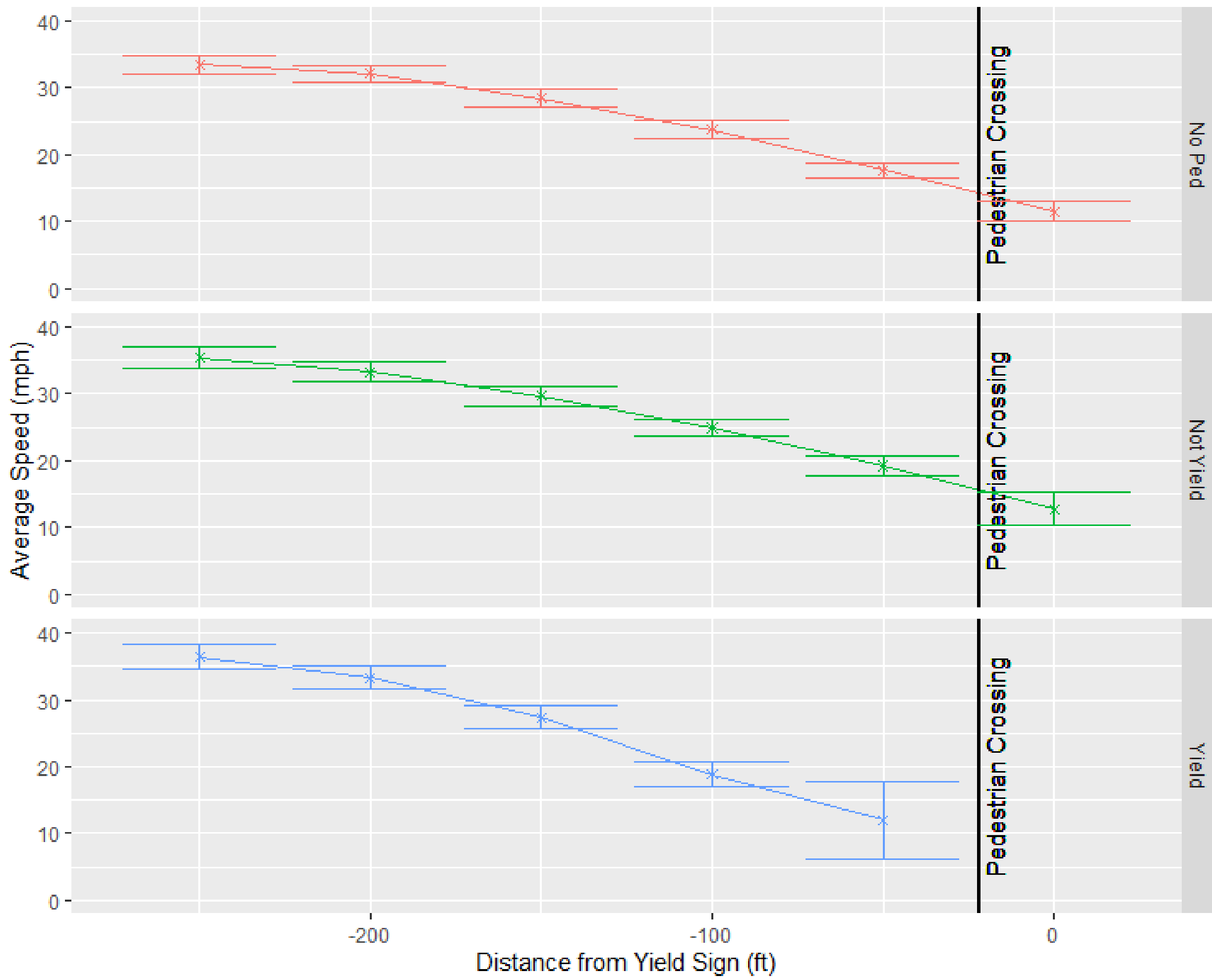

Figure 40 Interpolated Speed Profile at Every 50ft Interval for Site 16 NB Approach (M-53 at 26 Mile Road)

9 APPENDIX B

The following table shows site-by-site comparison of before and after annual average crash frequencies for each of the 180 roundabouts in the state of Michigan based on the raw crash data. The crash frequencies have not been normalized by traffic volume. The before and after annual average crash frequencies are presented based on total crashes, injury crashes (ABC), and KA crashes. The roundabouts are categorized based on number of circulating lanes.

Table 47 Site-by-Site Before and After Annual Average Crash Frequencies

Roundabout	Total Crashes		Injury Crashes		KA Crashes	
	Before	After	Before	After	Before	After
Single Lane Roundabouts						
25 Mile Rd E / Romeo Plank Rd.	4.00	3.33	0.75	0.67	0.08	0.00
3rd St & Western Ave	3.50	2.00	0.50	0.08	0.00	0.00
4th St. / Hickory Ave.	2.14	3.00	0.43	1.00	0.00	0.00
4th St. / West Ave.		2.38		0.25		0.00
7 Mile Rd & Brewer Ave	1.67	1.08	0.33	0.08	0.00	0.00
8 Mile Road & Whitmore Lake Road	6.69	6.00	1.00	1.00	0.00	0.00
Ann Arbor-Saline Rd. / Textile Rd.	3.75	8.67	0.58	1.67	0.08	0.33
Baker Rd. / Dan Hoey Rd.	3.50	8.00	0.50	0.00	0.00	0.00
Baker Rd. / Shield Rd. / Dongara Dr.	2.93	3.00	0.43	1.00	0.00	0.00
Barnes Ave. / Beal Ave.		0.60		0.07		0.00
Bennett Rd & Hulett Rd		3.80		0.73		0.07
Breton Rd. SE / Walma Ave. SE	4.63	3.29	0.88	0.14	0.25	0.00
Chambers/Renton & Johanna Ware	0.00	0.00	0.00	0.00	0.00	0.00
Cherry St & Jefferson Ave	3.67	2.42	1.67	0.33	0.00	0.08
Chesapeake Dr. / Walker Rd.		0.19		0.06		0.00
Chilson Rd. / E Coon Lake Rd.	3.08	3.00	0.77	0.00	0.23	0.00
Coachwood Ln. / Falcon Dr.		0.00		0.00		0.00
Community Dr. / Campus Pkwy.		0.50		0.06		0.00
Cooley Lake Rd & Bogie Lake Rd	4.33	5.67	1.33	0.50	1.00	0.08
Cooley Lake Rd. / Carroll Lake Rd.	7.07		1.47		0.07	
Cooper St. / I-94 WB Ramps	4.73		1.33		0.00	
Crescent Blvd. / Town Center Dr.	1.17	0.67	0.17	0.00	0.00	0.00
E Morrell St. / Martin Luther King Dr.	5.94		1.13		0.00	
East Rd. / Creyts Rd.		0.69		0.13		0.00
Elms Rd. / Hill Rd.	4.75	7.00	2.00	0.67	0.50	0.00

Roundabout	Total Crashes		Injury Crashes		KA Crashes	
	Before	After	Before	After	Before	After
Firewood Dr. / Raintree Dr.		0.56		0.13		0.00
Geddes Rd. / Earhart Rd.	13.00	16.50	4.00	1.88	0.14	0.13
Geddes Rd. / Ridge Rd.	3.22	4.33	0.56	0.67	0.00	0.00
Genesee Rd. / Davison Rd. / Lake Nepessing Rd.	15.18	7.75	1.82	1.25	0.09	0.25
Hamburg Rd & Winans Lake Rd	6.20	3.70	0.60	0.40	0.00	0.00
Hamilton Rd. / Marsh Rd.		7.13		0.75		0.00
Harding Ave. / Pershing Ave.	0.00	0.23	0.00	0.00	0.00	0.00
Harrison St. / E Ransom St. / E Gull St.	5.00	7.50	1.08	0.00	0.08	0.00
Hartland Rd. / Rovey Dr.	0.00	1.70	0.00	0.30	0.00	0.00
Hayes Rd & 25 Mile Rd	6.00	7.86	1.00	0.57	0.00	0.07
Howard St. / Solon St. / Kendall Ave.	5.67	10.00	0.67	1.00	0.33	0.11
I-75 NB off ramp & M-81/Washington Road	9.50	2.54	1.00	0.31	0.00	0.08
I-75 SB off ramp & M-81/Washington Rd	5.00	3.23	1.00	0.77	0.00	0.08
I-94 BR & 5th St	1.60	1.40	0.00	0.30	0.00	0.10
I-94 BR & Riverview Dr	5.40	2.90	1.20	0.30	0.00	0.00
I-94 EB Off Ramp & Main Street	1.86	3.38	0.29	0.50	0.14	0.00
Jefferson Ave & Wealthy St	12.50	12.00	2.50	1.09	0.00	0.09
Kensington Rd. / Jacoby Rd.	1.00	1.91	0.00	0.00	0.00	0.00
Kercheval Ave. / Wayburn St.	0.50	1.80	0.00	0.20	0.00	0.00
Kibby Rd. / Denton Rd.	2.86	2.00	0.21	0.00	0.07	0.00
Lakeshore Dr. / Monroe St. / Cottage Ln.		0.06		0.00		0.00
Lapeer Rd. / Allen Rd.	4.43	4.00	1.29	0.00	0.07	0.00
M-115 & M-37/ N 13 Road	7.00	3.20	1.10	0.00	0.40	0.00
M-186/State Street & US-131	7.00		2.27		0.47	
M-28 & Munising Ave	1.94		0.38		0.00	
M-43 & 72nd Street/CR 689 & 12th Avenue/CR 384	6.50	6.36	1.50	0.82	0.25	0.09
M-46/Apple Ave & M-37/Newaygo Rd	5.60	8.00	2.00	1.10	0.40	0.10
M-52 & Church/Broad Street	2.85	13.00	0.46	3.00	0.00	0.00
M-52 & Werkner Road	8.75	10.00	2.75	0.67	0.25	0.00
M-93 & Camp Grayling/ Howe Road	0.38	0.29	0.00	0.14	0.00	0.00
Main St & 3rd St		4.44		0.31		0.00
Maple Rd. / Skyline High School Entrance	0.67	0.25	0.33	0.08	0.00	0.08
Meijer Dr. / Greenville West Dr.		0.88		0.13		0.00
Michigan Ave & Washington Sq	9.67	13.67	0.67	0.67	0.00	0.00
Monroe Ave. NE / Guild St. NE	1.00	2.00	0.10	0.20	0.00	0.00
Monroe Ave. NE / Riverside Dr. NE / 3 Mile Rd. NE	0.80	1.20	0.10	0.20	0.00	0.00

Roundabout	Total Crashes		Injury Crashes		KA Crashes	
	Before	After	Before	After	Before	After
Moores River Dr. / Boston Blvd. / Pattengill Ave.	0.00	0.64	0.00	0.07	0.00	0.00
Mosher St. / Main St.	2.80	2.60	0.60	0.10	0.00	0.00
N 2nd St. / N 3rd St.	0.08	0.00	0.00	0.00	0.00	0.00
N Meridian Rd. / US 10 WB Ramps	6.36	3.25	1.18	1.00	0.27	0.25
N Park St. NE / Briggs Blvd. NE / Monroe Ave. NE	3.45	4.50	0.45	1.25	0.00	0.00
Napier Rd. / W 10 Mile Rd.	8.54	12.00	3.00	1.00	0.31	0.00
Nixon Rd & Huron Pkwy	4.00	5.20	0.80	0.70	0.00	0.10
Nixon Rd. / Dhu Varren Rd. / Green Rd.	3.85	2.50	0.77	0.00	0.00	0.00
Occidental Hwy. / Sutton Rd.	6.75		1.19		0.06	
Old 27 & Livingston Blvd	3.00	2.23	0.50	0.31	0.00	0.15
Oxbow Lake Rd & Cooley Lake Rd	1.00	1.67	0.00	0.00	0.00	0.00
Park Lake Rd. / Burcham Dr.	5.63	5.86	0.88	0.43	0.13	0.00
Pfeiffer Woods Dr. / (unknown)	0.00	0.33	0.00	0.08	0.00	0.00
Pioneer Dr. / Library Dr.	2.91	1.75	0.18	0.00	0.00	0.00
Pontiac Tr. / New Hudson Dr.	1.17	1.67	0.17	0.00	0.17	0.00
Presque Isle Ave. / Fair Ave.	2.46	2.50	0.46	1.00	0.00	0.00
Range Rd. / Griswold Rd.	6.33	3.33	1.83	0.67	0.17	0.33
Romeo Plank Rd. / Canal Rd.	8.17	8.33	2.17	1.67	0.33	0.44
Ryder Dr. / Connors Dr.	0.00	0.00	0.00	0.00	0.00	0.00
Ryder Dr. / Everett Dr.	0.00	0.07	0.00	0.00	0.00	0.00
Scio Church Rd. / S Wagner Rd.	6.77	11.00	0.92	2.50	0.00	0.50
Solon St. / Arboretum Pkwy.	0.17	5.22	0.00	0.11	0.00	0.00
St Anthony Rd. / US 23 NB Ramps	2.67	2.83	0.78	1.17	0.22	0.17
St Anthony Rd. / US 23 SB Ramps	4.33	2.67	0.33	0.50	0.11	0.00
Stevenson Ave. / Washington Ave. / Sheridan Rd.	2.67	2.33	0.67	0.44	0.00	0.00
Stratford Blvd & Charleston Dr/Plantation	0.00	0.21	0.00	0.07	0.00	0.00
Suncrest Dr. / Campus Pkwy.		0.38		0.00		0.00
Taft Rd. / Morgan Blvd.	0.60	1.30	0.20	0.10	0.20	0.00
Terrace Point Rd. / (parking lot access)		0.00		0.00		0.00
Texas Dr. / W Milham Rd. / S 12th St.	4.14	5.75	0.57	0.38	0.00	0.00
Textile Rd. / Hitchingham Rd.	5.27	11.00	1.36	1.25	0.09	0.00
Textile Rd. / Stony Creek Rd.	3.73	12.00	0.82	0.75	0.09	0.00
Tienken Rd. / Runyon Rd. / Washington Rd.		3.19		0.13		0.00
Tienken Rd. / Sheldon Rd.		7.31		0.81		0.00
US-10 BR/ M-20 & Patrick Road	4.10	2.80	1.90	0.40	0.10	0.00
US-12 & Old M-205/Five Points Road	6.83	5.33	2.08	1.00	0.67	0.33

Roundabout	Total Crashes		Injury Crashes		KA Crashes	
	Before	After	Before	After	Before	After
US-127 BR & Mission Rd	1.80	3.00	0.00	0.60	0.00	0.30
US-23 NB Off Ramp & 8 Mile Road	1.00	0.00	0.00	0.00	0.00	0.00
US-23 NB Off Ramp & N. Territorial Road	5.31	2.50	1.00	0.50	0.15	0.00
US-23 SB Off Ramp & 8 Mile Road	2.69	0.00	0.69	0.00	0.08	0.00
US-23 SB Off Ramp & N. Territorial Road	5.92	5.00	1.00	0.50	0.08	0.00
Village Place Blvd. / (parking lot access)	0.00	0.00	0.00	0.00	0.00	0.00
Village Place Blvd. / Green Oak Ave.	0.00	0.08	0.00	0.08	0.00	0.00
W Bemis Rd. / Moon Rd.	5.14	25.00	0.57	1.00	0.07	1.00
W Colon Rd. / Farrand Rd.	2.44		0.63		0.06	
W Main Ave. / Washington Ave.	1.27		0.13		0.00	
Waterside Dr & Vergote Dr	0.00	1.75	0.00	0.42	0.00	0.00
Wealthy St & Laffette Ave	9.00	6.82	1.25	1.36	0.25	0.18
White Lake Rd. / Duck Lake Rd. N	0.20	0.40	0.00	0.10	0.00	0.00
White Lake Rd. / Rose Center Rd. E	0.60	2.20	0.00	0.10	0.00	0.00
Whittaker Rd / Merrit Rd	3.33	2.67	1.00	0.00	0.08	0.00
Wiard Rd. / Airport Dr.	0.08	0.00	0.08	0.00	0.00	0.00
Willow Hwy & Canal Rd	3.50	4.82	0.25	0.45	0.00	0.00
Winchester St. / Devonshire St.	0.20		0.07		0.00	
Wright St. / Lincoln Ave.	3.85	3.00	0.62	0.00	0.08	0.00
Wright St. / Sugarloaf Ave.	7.54	6.00	1.92	1.00	0.00	0.00
Double Lane Roundabouts						
68th Ave & Randall St/State	8.67	12.50	1.67	1.00	0.00	0.00
Adams Rd. / Gunn Rd.	6.07		2.27		0.40	
Baldwin Rd & Coats/Indianwood Rd		7.00		0.67		0.07
Cedar St & Holbrook Dr	3.40	4.60	0.20	0.70	0.00	0.10
Civic Center Dr. / Evergreen Rd.	10.18	16.50	2.55	1.00	0.00	0.00
Evergreen Rd / (library and shopping center)	1.36	1.25	0.73	0.00	0.00	0.00
Geddes Rd & Superior Rd	10.75	9.64	2.75	1.00	0.50	0.00
Grand River Ave. / Lyon Center Dr. E	3.00	10.20	0.60	1.00	0.00	0.00
I-75BL & Mackinac Trail/3 Mile Road	12.67		2.33		0.47	
I-94 EB Off Ramp & Cooper Street	7.13		0.93		0.07	
I-94 EB Off Ramp & Sprinkle Road	5.18	59.00	0.91	2.25	0.00	0.50
I-94 WB Off Ramp & Main Street	7.14	6.75	1.43	0.50	0.00	0.00
I-94 WB Off Ramp & Sprinkle Road	29.00	38.00	4.18	2.50	0.00	0.00
Jefferson Ave. / Rosso Hwy.	11.33	3.33	2.92	1.00	0.42	0.00
Lake Lansing Rd & Chamberlain Dr	3.67	6.92	0.67	0.75	0.00	0.17

Roundabout	Total Crashes		Injury Crashes		KA Crashes	
	Before	After	Before	After	Before	After
Loop Rd. / Commerce Crossing Dr.	0.00	3.23	0.00	0.31	0.00	0.00
M-11 & Remembrance Road	10.18	35.75	3.36	4.75	0.55	0.25
M-14 EB & Maple Rd	4.67	10.67	1.67	1.08	0.00	0.17
M-14 WB & Maple Rd	11.00	6.83	2.33	0.58	0.00	0.00
M-53 NB off ramp & 26-Mile	13.40	19.70	2.60	1.60	0.40	0.00
M-53 SB off ramp & 26-Mile	12.20	17.00	2.80	1.70	0.20	0.00
M-72 & Lautner Road	3.00	23.75	0.64	1.25	0.18	0.25
M-72 / Town Center Dr.	0.64	3.75	0.00	0.75	0.00	0.25
Martin Pkwy. / Library Dr.	0.00	5.20	0.00	0.40	0.00	0.30
Martin Pkwy. / Oakley Park Rd.	7.40	41.60	2.00	1.50	0.20	0.20
Martin Pkwy. / PGA Dr.	0.00	6.60	0.00	0.40	0.00	0.00
Michigan Ave & Rankin Ave		4.27		0.47		0.00
NB I-75 Off Ramp & Bristol Road	16.50	18.00	3.92	0.67	0.08	0.00
Romeo Plank Rd & 19 Mile Rd	18.00	35.45	3.75	2.45	0.25	0.18
Romeo Plank Rd & Cass Ave	8.50	30.18	2.00	2.27	0.25	0.18
S Baldwin Rd. / Gregory Rd.	7.86	18.00	0.79	1.00	0.14	0.00
S Baldwin Rd. / Judah Rd.	5.79	10.00	0.64	2.00	0.00	1.00
State St. / Ellsworth Rd.	21.89	133.33	3.00	4.67	0.00	0.17
US-127 BR & N. Mission Road	8.69	3.50	1.62	0.00	0.38	0.00
US-23 NB Off Ramp & Geddes Road	6.50	7.44	0.83	1.00	0.00	0.00
US-23 SB Off Ramp & Geddes Road	7.00	7.33	1.50	0.89	0.00	0.11
US-41 & Brickyard Road	6.60		1.60		0.20	
US-41 & Grove Street/7th St	10.54	15.00	2.31	1.00	0.23	0.00
US-41 & Marquette Hospital Drive	0.31	1.50	0.08	0.00	0.00	0.00
US-41/M-28 & 2nd Street	8.42	17.33	2.92	1.00	0.75	0.00
US-41/M-28 & Front Street	8.83	9.22	2.17	1.22	0.00	0.00
Utica Rd & Dodge Park Rd	17.40	15.90	1.20	2.10	0.00	0.10
W 14 Mile Rd. / Orchard Lake Rd.	58.27	159.50	13.64	19.50	0.45	0.00
W Baraga Ave / Hospital Dr. / (hospital)	0.08	0.00	0.00	0.00	0.00	0.00
W Hamlin Rd. / S Livernois Rd.	11.67	57.22	2.50	4.56	0.17	0.00
W Maple Rd. / Middlebelt Rd.	35.80		7.60		0.20	
W Tienken Rd. / N Livernois Rd.	14.60	29.00	1.90	1.60	0.10	0.20
Whittaker Rd. / Stony Creek Rd.	14.67	10.33	3.17	1.78	0.17	0.00
Wood St & Sam's Way	2.00	6.00	0.33	0.92	0.33	0.42
Triple Lane Roundabouts						
Farmington Rd & 14 Mile Rd	36.25	66.82	7.50	6.45	0.50	0.09

Roundabout	Total Crashes		Injury Crashes		KA Crashes	
	Before	After	Before	After	Before	After
M-5 & Pontiac Trail	33.86	148.38	6.86	8.88	0.29	0.25
M-53 at 18 ½ Mile (Van Dyke) Road	13.00	141.21	1.00	10.57	0.00	0.14
Maple Rd & Farmington Rd	42.00	73.17	7.33	7.25	0.00	0.25
Maple Rd & Drake Rd	23.33	56.67	4.67	5.42	0.00	0.00
US-23 NB & Lee Rd	2.00	8.69	0.00	0.54	0.00	0.00
US-23 SB & Lee Rd	4.00	9.31	0.50	0.23	0.00	0.00
Whitmore Lake Rd & Lee Rd	17.00	58.00	4.00	3.46	1.00	0.23

10 APPENDIX C

This appendix presents the detailed calculations on crash costs estimation and benefit-cost ratio estimation on a site-by-site basis. The crash costs are estimated before and after roundabout conversion based on two methods-

- Raw crash counts
- Empirical Bayes estimates

Table 48 and Table 49 present the site-by-site crash frequencies in the pre- and post-construction period and the associated change in crash costs, respectively, based on the observed raw crash counts. Similar results are presented based on EB estimated crash frequencies in Table 50 and Table 51. Suitable EB estimates are selected from Table 27. Finally, the site-by-site calculation for benefit/cost ratios are presented in Table 52.

Table 48 Site-by-Site Crash Frequencies based on Raw Crash Counts

Site	Pre-Construction						Post-Construction					
	K	A	B	C	PDO	Years	K	A	B	C	PDO	Years
US-23 NB Off Ramp & Geddes Road	0	0	0	5	34	6	0	0	2	7	58	9
US-23 SB Off Ramp & Geddes Road	0	0	2	7	33	6	0	1	2	5	58	9
US-23 SB & Lee Rd	0	0	0	1	7	2	0	0	1	2	118	13
M-52 & Werkner Road	0	3	12	18	72	12	0	0	2	0	28	3
US-127 BR & Mission Rd	0	0	0	0	9	5	0	3	0	3	24	10
M-5 & Pontiac Trail	0	2	9	37	189	7	0	2	14	55	1116	8
NB I-75 Off Ramp & Bristol Road	0	1	10	36	151	12	0	0	0	2	52	3
I-94 EB Off Ramp & Sprinkle Road	0	0	1	9	47	11	0	2	3	4	227	4
I-94 WB Off Ramp & Sprinkle Road	0	0	12	34	273	11	0	0	3	7	142	4
US-41/M-28 & Front Street	0	0	0	13	40	6	0	0	2	9	72	9
US-10 BR/ M-20 & Patrick Road	0	1	6	12	22	10	0	0	0	2	12	5
I-75 NB off ramp & M-81/Washington Road	0	0	0	2	17	2	0	1	1	2	29	13
M-53 NB off ramp & 26-Mile	1	1	3	9	53	5	0	0	1	15	181	10
M-11 & Remembrance Road	0	6	11	20	75	11	0	1	1	17	124	4
Maple Rd & Farmington Rd	0	0	2	20	104	3	0	3	11	73	791	12

Table 49 Change in Crash Costs based on Raw Crash Counts

Site	K-Cost	A-Cost	B-Cost	C-Cost	PDO-Cost	Total Cost
US-23 NB Off Ramp & Geddes Road	-	-	$47,551.78	$(7,522.04)	$9,977.49	$50,007
US-23 SB Off Ramp & Geddes Road	-	$78,454.44	$(23,775.89)	$(82,742.49)	$12,115.52	$(15,948)
US-23 SB & Lee Rd	-	-	$16,460.23	$(46,868.12)	$71,541.88	$41,134
M-52 & Werkner Road	-	$(176,522.50)	$(71,327.67)	$(203,095.20)	$42,760.67	$(408,185)
US-127 BR & Mission Rd	-	$211,827.00	-	$40,619.04	$7,696.92	$260,143
M-5 & Pontiac Trail	-	$(25,217.50)	$99,349.25	$215,184.20	$1,443,172.50	$1,732,488
NB I-75 Off Ramp & Bristol Road	-	$(58,840.83)	$(178,319.17)	$(315,925.87)	$60,933.95	$(492,152)
I-94 EB Off Ramp & Sprinkle Road	-	$353,045.00	$141,034.25	$24,617.60	$673,188.95	$1,191,886
I-94 WB Off Ramp & Sprinkle Road	-	-	$(72,948.75)	$(181,554.80)	$137,028.50	$(117,475)
US-41/M-28 & Front Street	-	-	$47,551.78	$(157,962.93)	$17,104.27	$(93,307)
US-10 BR/ M-20 & Patrick Road	-	$(70,609.00)	$(128,389.80)	$(108,317.44)	$2,565.64	$(304,751)
I-75 NB off ramp & M-81/Washington Road	-	$54,314.62	$16,460.23	$(114,566.52)	$(80,422.95)	$(124,215)

Site	K-Cost	A-Cost	B-Cost	C-Cost	PDO-Cost	Total Cost
M-53 NB off ramp & 26-Mile	$(2,435,288)	$141,218.00)	$(106,991.50)	$(40,619.04)	$96,211.50	$(2,627,905)
M-11 & Remembrance Road	-	$(208,617.50)	$(160,487.25)	$329,260.40	$310,209.20	$270,365
Maple Rd & Farmington Rd	-	$176,522.50	$53,495.75	$(78,981.47)	$400,881.25	$551,918

Table 50 Site-by-Site Crash Frequencies based on EB Estimates

Site	Pre-Construction					EB Estimate	EB Estimate	Post-Construction				
	K	A	B	C	PDO	KABC	PDO	K	A	B	C	PDO
US-23 NB Off Ramp & Geddes Road	0.00	0.00	0.00	0.83	5.67	0.81	1.92	0.00	0.00	0.00	-0.16	5.21
US-23 SB Off Ramp & Geddes Road	0.00	0.00	0.33	1.17	5.50	0.81	1.92	0.00	0.00	-0.06	-0.22	5.06
US-23 SB & Lee Rd	0.00	0.00	0.00	0.50	3.50	1.04	2.27	0.00	0.00	0.00	0.02	4.45
M-52 & Werkner Road	0.00	0.25	1.00	1.50	6.00	0.81	1.92	0.00	-0.05	-0.19	-0.29	5.52
US-127 BR & Mission Rd	0.00	0.00	0.00	0.00	1.80	0.76	1.29	0.00	0.00	0.00	0.00	0.52
M-5 & Pontiac Trail	0.00	0.29	1.29	5.29	27.00	1.07	2.28	0.00	0.02	0.09	0.37	34.56
NB I-75 Off Ramp & Bristol Road	0.00	0.08	0.83	3.00	12.58	0.81	1.92	0.00	-0.02	-0.16	-0.57	11.58
I-94 EB Off Ramp & Sprinkle Road	0.00	0.00	0.09	0.82	4.27	0.81	1.92	0.00	0.00	-0.02	-0.16	3.93
I-94 WB Off Ramp & Sprinkle Road	0.00	0.00	1.09	3.09	24.82	0.76	1.29	0.00	0.00	-0.26	-0.74	7.20
US-41/M-28 & Front Street	0.00	0.00	0.00	2.17	6.67	0.76	1.29	0.00	0.00	0.00	-0.52	1.93
US-10 BR/ M-20 & Patrick Road	0.00	0.10	0.60	1.20	2.20	0.81	1.92	0.00	-0.02	-0.11	-0.23	2.02
I-75 NB off ramp & M-81/Washington Road	0.00	0.00	0.00	1.00	8.50	0.81	1.92	0.00	0.00	0.00	-0.19	7.82
M-53 NB off ramp & 26-Mile	0.20	0.20	0.60	1.80	10.60	0.76	1.29	-0.05	-0.05	-0.14	-0.43	3.07
M-11 & Remembrance Road	0.00	0.55	1.00	1.82	6.82	0.81	1.92	0.00	-0.10	-0.19	-0.35	6.27
Maple Rd & Farmington Rd	0.00	0.00	0.67	6.67	34.67	1.07	2.28	0.00	0.00	0.05	0.47	44.37

Table 51 Change in Crash Costs based on EB Estimates

Site	K-Cost	A-Cost	B-Cost	C-Cost	PDO-Cost	Total Cost
US-23 NB Off Ramp & Geddes Road	-	-	-	$(21,437.83)	$66,877.68	$45,439.86
US-23 SB Off Ramp & Geddes Road	-	-	$(13,552.26)	$(30,012.96)	$64,910.69	$21,345.48
US-23 SB & Lee Rd	-	-	-	$2,707.94	$57,021.35	$59,729.29
M-52 & Werkner Road	-	$(33,539.28)	$(40,656.77)	$(38,588.09)	$70,811.66	$(41,972.47)
US-127 BR & Mission Rd	-	-	-	-	$6,696.32	$6,696.32
M-5 & Pontiac Trail	-	$14,121.80	$19,258.47	$50,096.82	$443,342.59	$526,819.68
NB I-75 Off Ramp & Bristol Road	-	$(11,179.76)	$(33,880.64)	$(77,176.18)	$148,507.80	$26,271.22
I-94 EB Off Ramp & Sprinkle Road	-	-	$(3,696.07)	$(21,048.05)	$50,426.49	$25,682.37
I-94 WB Off Ramp & Sprinkle Road	-	-	$(56,024.64)	$(100,439.81)	$92,328.05	$(64,136.39)
US-41/M-28 & Front Street	-	-	-	$(70,406.34)	$24,801.19	$(45,605.15)
US-10 BR/ M-20 & Patrick Road	-	$(13,415.71)	$(24,394.06)	$(30,870.47)	$25,964.28	$(42,715.97)
I-75 NB off ramp & M-81/Washington Road	-	-	-	$(25,725.39)	$100,316.52	$74,591.13
M-53 NB off ramp & 26-Mile	$(584,469.18)	$(33,892.32)	$(30,813.55)	$(58,491.42)	$39,433.89	$(668,232.58)
M-11 & Remembrance Road	-	$(73,176.60)	$(40,656.77)	$(46,773.44)	$80,467.80	$(80,139.01)
Maple Rd & Farmington Rd	-	-	$9,985.87	$63,185.17	$569,229.99	$642,401.04

Table 52 Site-by-Site Benefit/Cost Ratio

Site	Annual Crash Costs (Raw Count)	Annual Crash Costs (EB Estimate)	Annual Delay Savings	Annual Fuel Savings	Equivalent Annualized Construction Costs	B/C (Raw Crash Count)	B/C (EB Estimate)
US-23 NB Off Ramp & Geddes Road	$50,007.22	$45,439.86	$(347,205.72)	$53,705.65	$144,575.25	-2.38	-2.34
US-23 SB Off Ramp & Geddes Road	$(15,948.41)	$21,345.48	$(502,535.97)	$52,465.65	$144,575.25	-3.00	-3.26
US-23 SB & Lee Rd	$41,133.99	$59,729.29	$1,121,238.83	$50,943.21	$173,490.30	6.52	6.41
M-52 & Werkner Road	$(408,184.70)	$(41,972.47)	$276,729.04	$38,811.90	$115,660.20	6.26	3.09
US-127 BR & Mission Rd	$260,142.96	$6,696.32	$150,410.70	$22,152.32	$115,660.20	-0.76	1.43
M-5 & Pontiac Trail	$1,732,488.45	$526,819.68	$585,478.54	$125,976.80	$173,490.30	-5.89	1.06

Site	Annual Crash Costs (Raw Count)	Annual Crash Costs (EB Estimate)	Annual Delay Savings	Annual Fuel Savings	Equivalent Annualized Construction Costs	B/C (Raw Crash Count)	B/C (EB Estimate)
NB I-75 Off Ramp & Bristol Road	$(492,151.92)	$26,271.22	$1,147,328.93	$61,492.37	$144,575.25	11.77	8.18
I-94 EB Off Ramp & Sprinkle Road	$1,191,885.80	$25,682.37	$427,645.14	$92,694.37	$144,575.25	-4.64	3.42
I-94 WB Off Ramp & Sprinkle Road	$(117,475.05)	$(64,136.39)	$370,102.89	$80,951.14	$144,575.25	3.93	3.56
US-41/M-28 & Front Street	$(93,306.89)	$(45,605.15)	$1,299,793.78	$83,759.50	$144,575.25	10.22	9.89
US-10 BR/ M-20 & Patrick Road	$(304,750.60)	$(42,715.97)	$31,063.88	$24,912.46	$115,660.20	3.12	0.85
I-75 NB off ramp & M-81/Washington Road	$(124,214.62)	$74,591.13	$194,114.00	$45,625.00	$115,660.20	3.15	1.43
M-53 NB off ramp & 26-Mile	$(2,627,905.28)	$(668,232.58)	$2,287,957.47	$144,241.50	$144,575.25	35.00	21.45
M-11 & Remembrance Road	$270,364.85	$(80,139.01)	$1,379,701.76	$44,991.22	$144,575.25	7.98	10.41
Maple Rd & Farmington Rd	$551,918.03	$642,401.04	$3,699,139.58	$82,742.24	$173,490.30	18.62	18.10
Total	$(85,996.16)	$486,174.81	$12,120,962.86	$1,005,465.33	$2,139,713.67	6.17	5.91