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Evaluation of Alternative Intersections and Interchanges

Volume I — Roundabout Capacity and Rollover Analysis for Heavy Vehicles



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| 16. Abstract <p>There is a recent trend of building roundabouts on high-speed roads, often with the considerable presence of heavy vehicles. With the increased presence of trucks on roundabouts, the issue of overturning has become a concern. Although some geometric, vehicle, and loading factors have been connected to rollover, the safety performance of roundabouts built on high-speed roads is not well understood due to their relative novelty. In addition, other concerns related to geometry, driver behavior, and environmental considerations may exist at roundabouts. This study examined roundabout circulatory superelevation, aggressive driver behavior, roundabout readability, and nighttime conditions in the context of heavy vehicle rollover. Moreover, the critical and follow-up headways were estimated for trucks and other vehicles at roundabouts located on the low- and high-speed roads and during daytime and nighttime conditions.</p> <p>This research developed a methodology which may be used to examine truck overturning at roundabouts. A generalized rollover model suitable for application to heavy vehicles was applied to field-observed semi-trailer speeds and paths to estimate their proximity to rollover at newly-built Indiana roundabouts. This was done by introducing Δv - the difference between the critical rollover speed determined from the model and the actual speed.</p> <p>This report revealed that heavy vehicles increased the critical headway, and in turn reduced the entry capacity of roundabouts. Drivers of heavy vehicles, on average, accepted a 1.1 sec longer critical headway than drivers of passenger cars. The effects of nighttime/twilight conditions indicated additional capacity reduction caused by a 0.6 sec longer critical headway compared to daylight conditions. Likewise, drivers on dual-lane roundabouts in rural areas accepted a 0.6 sec longer critical headway than drivers on single-lane roundabouts in urban areas.</p> <p>It was determined that the gap-acceptance parameters for a single-lane roundabout on a low-speed state road were shorter than the national values, resulting on average in 30% higher capacity for Indiana conditions. In contrast, the estimated critical headway was larger for dual-lane roundabouts on high-speed state roads, resulting in 15% reduced capacity for Indiana conditions.</p> <p>The findings of this report are based on low and medium traffic volumes presently observed on high-speed rural and suburban roads. Heavy traffic flow may affect driver behavior; therefore, studying such roundabouts in heavier traffic conditions might improve the results.</p> | | | |
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EXECUTIVE SUMMARY

EVALUATION OF ALTERNATIVE INTERSECTIONS AND INTERCHANGES: VOLUME I—ROUNDBOUT CAPACITY AND ROLLOVER ANALYSIS FOR HEAVY VEHICLES

Introduction

There is a recent trend of building roundabouts on high-speed roads, often with the considerable presence of heavy vehicles. With the increased presence of trucks on roundabouts, the issue of overturning has become a concern. Although some geometric, vehicle, and loading factors have been connected to rollover, the safety performance of roundabouts built on high-speed roads is not well understood. This study compared the heavy vehicle rollover risk for roundabouts on low- and high-speed roads, while also examining roundabout circulatory superelevation, aggressive driver behavior, roundabout readability, and nighttime conditions in the context of rollover. Moreover, the critical and follow-up headways were estimated for trucks and other vehicles at roundabouts located on the low- and high-speed roads and during daytime and nighttime conditions.

Findings

This research developed a methodology which was used to examine truck overturning at roundabouts. A generalized rollover model suitable for application to heavy vehicles was applied to field-observed semi-trailer speeds and paths to estimate their proximity to rollover at newly built Indiana roundabouts. This was done by introducing Δv , the difference between the critical rollover speed determined from the model and the actual speed.

The research detected no excessive rollover risk on the studied roundabout built on the high-speed road. The benefit of an inward-sloped circulatory roadway was too small to justify its introduction to design practice. High speeds in advance of a roundabout, associated with aggressive driver behavior, did not result in a considerable increase in the rollover propensity at the roundabout. Although a larger deceleration rate on the roundabout approach was associated with a slightly higher rollover risk, a large safety margin was still preserved. Night conditions did not

bring any increase in the propensity for rollover. Driver behavior tended to be more cautious under night conditions than during the day. A wider circulatory roadway may be associated with a lower rollover propensity by allowing drivers to compensate for higher speeds with a flatter path. An examination of literature and crash reports found that a cautious design of the truck apron is warranted. It should be easily mountable and marked conspicuously with texture and color different from the pavement.

This report revealed that heavy vehicles increased the critical headway and, in turn, reduced the entry capacity of roundabouts. Drivers of heavy vehicles, on average, accepted a 1.1 sec longer critical headway than drivers of passenger cars. The effects of nighttime/twilight conditions indicated additional capacity reduction caused by a 0.6 sec longer critical headway compared to daylight conditions. Likewise, drivers on dual-lane roundabouts in rural areas accepted a 0.6 sec longer critical headway than drivers on single-lane roundabouts in urban areas. It was determined that the gap-acceptance parameters for a single-lane roundabout on a low-speed state road were shorter than the national values, on average resulting in 30% higher capacity for Indiana conditions. In contrast, the estimated critical headway was larger for dual-lane roundabouts on high-speed state roads, resulting in 15% reduced capacity for Indiana conditions.

The findings of this report are based on low and medium traffic volumes presently observed on high-speed rural and suburban roads. Heavy traffic flow may affect driver behavior; therefore, studying such roundabouts in heavier traffic conditions might improve the results.

Implementation

The propensity for rollover at the studied roundabouts during the observation period was low. No considerable difference in the rollover propensity between the studied roundabouts on low- and high-speed roads was found. This finding does not provide a basis for recommending changes in the current design policy for roundabouts. It should be noted, though, that Indiana is in the early phase of introducing roundabouts on high-speed roads.

The estimated critical and follow-up headways may be used instead of the default national values in capacity and LOS evaluation by INDOT designers and traffic engineers.

The limited number of roundabouts available in this study prompts for a similar study in the future.

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1. INTRODUCTION

1.1 Background

Alternative intersections and interchanges are becoming more prevalent across the United States for the replacement of traditional intersection designs. A number of types have emerged, including single point and diverging diamond interchanges, median U-turn (Michigan left), continuous flow, and roundabouts. Roundabouts are predominantly used due to their safety and capacity benefits. Around 3200 now exist throughout the United States, with the largest concentrations in Washington, Wisconsin, and Florida (Rodegerdts, 2014). As of 2013, Indiana had nearly 100 roundabouts built by the state or municipalities (INDOT, 2014; Rodegerdts, 2014). The Indiana Department of Transportation (INDOT) installed its first roundabout in 2008 in Valparaiso and has planned nearly 30 additional roundabouts on state roads by 2017.

According to National Cooperative Highway Research Program (NCHRP) report 672, a 76% reduction in injury crashes and a 35% drop in all crashes was found in a nationwide roundabout study (Rodegerdts, Bansen, et al., 2010). The converted intersections had previously been controlled as two way stops, all way stops, or signalized. Similar crash reductions have been seen in European countries (Jensen, 2013). Benefits of installing roundabouts can be attributed to a variety of factors, including fewer and less severe conflict points, lower speeds, and enhanced pedestrian safety (Rodegerdts, Bansen, et al., 2010). They are also known to reduce queuing and the delays faced by drivers, thus allowing better traffic progression than conventional intersections when volumes are not exceptionally large.

A question that needs to be better answered before roundabouts can be confidently built is how they will perform on high-speed roadways. The speed threshold of 45 mph is commonly used to separate low from high-speed roadways. High-speed conditions exist on the edges of towns and cities where there is a need for roundabouts to transition from a high-speed rural environment to lower speed urban roads (Torbic et al., 2012). Roundabout safety examinations on these types of roads have been rather brief, but show consistency with results from lower speed roads in reducing accidents, particularly those that are most severe (Bill, Qin, Chitturi, & Noyce, 2011; Isebrands, 2011). Figure 1.1 shows such a roundabout in Kansas, where posted speeds on the approaches can be as high as 65 mph.

Although roundabouts can reduce overall crashes, there is a safety concern emerging for truck rollover as more and more roundabouts are built on high-speed roadways where the presence of heavy vehicles tends to be considerable. Kansas has considerable experience in this area. Since 2000, half of the heavy vehicle crashes at roundabouts on high-speed roads have been rollovers.



Figure 1.1 US-400 and K-47 roundabout near Fredonia, KS (Google Earth).

The common factor among these accidents was speed excessive for the conditions.

Despite restrictions on heavy vehicles on many local roundabouts, the United Kingdom observes 50–60 injury rollovers per year on roundabouts (Highways England, 2007). An examination of 100 urban and rural roundabouts in Queensland, Australia found articulated vehicles “overrepresented in the single-vehicle accident data” due to their tendency to roll (Arndt & Troutbeck, 1998). Truck rollover at roundabouts is an issue many agencies seek to better understand and address.

Geometric features that allow excessive speed on the approach and entry have been connected to rollover, as well as sudden changes in cross fall and radius (Highways England, 2007). However, research has not quantified the proximity to rollover for heavy vehicles and how factors such as approach high speed and environmental conditions affect this threshold. To investigate this topic, an improved model of the rollover conditions applicable to heavy vehicles at roundabouts is needed.

Furthermore, it is important to check if the existing capacity models may be applied to roundabouts at high-speed location on arterial rural and suburban highways to ensure proper design and sufficient capacity of such roundabouts. Several empirical and analytical capacity models are available for roundabouts, including the well-known Highway Capacity Manual (HCM) 2010 model developed for U.S. conditions (Rodegerdts, Blogg, et al., 2007).

Two main parameters used in gap-acceptance models are the critical headway (critical gap) and the follow-up headway (follow-up time). Critical headway is the

shortest time headway between two consecutive vehicles on circulatory roadways that is acceptable to an average driver waiting to enter the roundabout safely. However, a distinction between “gap” and “headway” is important. A gap represents the time difference that the rear bumper of the leading vehicle clears the conflict line and the front bumper of the following vehicle occupies that line, whereas, headway represents time difference between the front-to-front bumpers. In this report, the term headway is used rather than that of gap. The follow-up headway is the average time headway between consecutive vehicles on the approach roadways entering the roundabout from a queue by accepting the same available headway in the circulatory traffic. Although default values for these parameters are reflected in the HCM 2010, the values are not applicable to all conditions. HCM recommends calibrating the gap-acceptance parameters for local conditions.

A limited number of research studies have been conducted on rural roundabouts in the U.S. The largest collection of roundabout data in the U.S., in existence since 2003, contains 90 percent of the data from urban and suburban areas (Rodegerdts, Blogg, et al., 2007). This database was used for developing the HCM 2010 capacity model. In addition, only a few past studies on Indiana roundabouts have taken place, which were located in urban/suburban areas in Carmel, Indiana (Day, Hainen, & Bullock, 2013; Tarko, Inerowicz, & Lang, 2008; Wei & Grenard, 2012). Carmel has been building roundabouts since the late 1990s and drivers are accustomed to them, unlike drivers elsewhere in Indiana. Therefore, the capacity-related findings obtained through these studies may not be transferable to larger roundabouts with high-speed approaches on Indiana state roads.

Moreover, the previous studies for Indiana roundabouts did not address dual-lane roundabouts or the effects of heavy vehicles (single-unit truck, bus, and semi-trailer) on roundabout capacity. Also, none of the studies addressed the effects of lighting conditions (nighttime/twilight in the presence of street lighting) as rush hour happens at twilight and relatively dark conditions during late fall and early winter. Therefore, this report examines roundabouts built on state roads in Indiana as well as the factors that affect their operational performance.

1.2 Scope of Work and Research Objectives

The scope of work described in this report includes examination of previous studies and roundabout crash statistics. A rollover model, that represents the rollover conditions of trucks on tight curves better than current models, will be developed and applied to assess the rollover propensity at Indiana roundabouts built on high-speed and low-speed roads. This study addresses the following safety-related topics:

- (1) Discern whether there is a significant difference in the rollover propensity at roundabouts on high-speed versus low-speed roads.

- (2) Examine whether inward circulatory superelevation affords considerable safety advantages over the commonly used outward design.
- (3) Determine if aggressive driver behavior, as displayed by high speeds far from the roundabout, suggest drivers are more likely to encroach on critical rollover conditions at the roundabout.
- (4) Determine whether strong braking near the roundabout, which may be indicative of a driver misreading the roundabout geometry, is associated with a greater rollover risk.
- (5) Examine how nighttime conditions, common in northern latitudes during the late fall and winter months, affect the rollover threshold at the roundabout.

Furthermore, patterns in the crash records will be examined as they provide important insight into the actual causes of rollover.

For the capacity analysis, the research objective is to evaluate the capacity of modern roundabouts built on high-speed roads. Specifically, the research’s aim is to identify the factors that affect the gap-acceptance behaviors of drivers on roundabouts built on high-speed Indiana state highways in rural areas. The effects of high-speed approaches and heavy vehicles on roundabout capacity are studied as well as the effects of nighttime/twilight conditions on drivers. The results are intended to improve the capacity analysis of roundabouts designed on Indiana state roads and to contribute to an increased understanding of capacity factors in general.

1.3 Report Organization

This remainder of this report is organized into the following chapters:

- Chapter 2 Literature Review
- Chapter 3 Research Method
- Chapter 4 Data
- Chapter 5 Rollover Propensity Analysis
 - Low and High-Speed Roads
 - Circulatory Superelevation
 - Aggressive Behavior
 - Roundabout Readability
 - Nighttime Conditions
- Chapter 6 Capacity Analysis
- Chapter 7 Conclusions
- Appendices

2. LITERATURE REVIEW

2.1 Introduction

This chapter reviews roundabout safety, as well as factors affecting safety, first generally for roundabouts and then with a focus on those installed on high-speed roads. Heavy vehicle rollover and the factors influencing it are discussed, particularly in the context of roundabouts. The capacity analysis provides an overview of current empirical and analytical methods, as well as simulation models that can be used as an

alternative to such methods. Studies on gap acceptance and factors influencing it are examined. Gaps in knowledge are identified and provide the framework for the rest of the report.

2.2 Safety Background

2.2.1 Crash Statistics

Roundabouts have a good record of decreasing the number of severe crashes. Persaud, Retting, Garder, and Lord (2001) observed improvements in injury (80%) and total crashes (40%) for US roundabouts. Fatal and incapacitating injury crashes were nearly eliminated, a trend echoed in Wisconsin (Bill et al., 2011) and Maryland (Rice & Niederhauser, 2010).

Internationally, Europe has the most roundabouts by a wide margin. An analysis at 332 Danish intersections converted to roundabouts revealed decreases in injury (47%) and PDO crashes (16%) (Jensen, 2013). More significant safety improvements were observed for roundabouts located on high-speed roads. The United Kingdom and France have the most roundabouts: 25,000 and 32,000, respectively (Baranowski, 2014).

A summary of the observed crash reductions in these countries and others after building a roundabout are presented in Table 2.1.

2.2.2 Roundabout Geometric Factors Affecting Crash Rates

The effect of certain roundabout geometric factors on accident rates has been examined. One of the first studies from the United Kingdom found that the entry width and entry path curvature are significant (Maycock & Hall, 1984). Research was later extended to 100 urban and rural roundabouts in Queensland, Australia (Arndt & Troutbeck, 1998). Factors affecting both single and multiple-vehicle accident rates were studied. About 18% of accidents involved a single-vehicle. Lengthy curves with heavily-used side friction, high absolute speed on elements, and significant speed reductions between elements increased the crash rate. The majority of accidents occurred in the circulation. Articulated vehicles were overly represented due to their rollover propensity. The remaining 82% of crashes

TABLE 2.1
International Mean Crash Reductions at Roundabouts.

| Country | Reduction | |
|----------------|-----------------|--------------------|
| | All Crashes (%) | Injury Crashes (%) |
| Australia | 41–61 | 45–87 |
| France | — | 57–78 |
| Germany | 36 | — |
| Netherlands | 47 | — |
| United Kingdom | — | 25–39 |

Source: Robinson et al. (2000).

involved multiple vehicles. Poor visibility and speed difference between motorists increased the rate. Geometric features known to affect crash rates are shown in Table 2.2.

Single and multilane roundabouts are common in the United States. The most common crash type among single lane roundabouts are entering-circulating accidents, due to an inability of entering drivers to predict the behavior of circulating drivers (Zheng, Qin, Tillman, & Noyce, 2013). Multilane roundabouts introduce other conflict types, including turns from improper lanes and lane changing within the roundabout (Hourdos & Richfield, 2014). Although every accident pattern tends to increase at multilane roundabouts, the increase is largest for sideswipe accidents (Zheng et al., 2013).

2.3 High-Speed Conditions

2.3.1 Crash Statistics

Roundabouts have traditionally been built on low-speed roads, but they are becoming more prevalent on high-speed roads. Safety examinations have been rather cursory at these roundabouts. A five-state study of rural roundabouts found 88% and 63% reductions in injury and total accidents, respectively (Isebrands, 2011). Research from high-speed intersections converted to roundabouts in Wisconsin showed a 30% drop in total accidents and elimination of fatal accidents (Bill et al., 2011). Table 2.3 summarizes the results and highlights the trend of larger improvements for the more severe crash types.

2.3.2 Roundabout Design on High-Speed Roads

From the roundabout design perspective, drivers must be adequately warned so they may reduce their speeds. In this regard, studies have compared roundabouts with more traditional intersection controls, such as stop signs. Isebrands, Hallmark, and Hawkins (2014) studied roundabouts and two-way stop-controlled

TABLE 2.2
Geometric Factors Affecting Crash Rates.

| Geometric Factor | Effect on: | |
|-------------------------------------|------------------------------|-----------------------------|
| | Entering/Circulating Crashes | Exiting/Circulating Crashes |
| Increased entry width | Increase | — |
| Increased central island diameter | Decrease | Increase |
| Increased angle between legs | Decrease | — |
| Increased inscribed circle diameter | — | Increase |
| Increased circulating width | — | Increase |
| Increased lane width | Increased approach crashes | |

Source: Based on Rodegerdts, Bansen, et al. (2010).

TABLE 2.3
Crash Reductions at Roundabouts on High-Speed Roads in Wisconsin.

| Crash Type | % Decrease |
|---|------------|
| Fatal | 100% |
| Incapacitating injury | 75% |
| Non-incapacitating injury | 60% |
| Possible injury | 67% |
| Property damage only | 9% |
| Overall (121 crashes before, 85 crashes after) | 30% |

Source: Bill et al. (2011).

intersections in Iowa, Kansas, and Minnesota. Table 2.4 provides a speed comparison at different distances from the yield line/stop bar.

Roundabouts, at least those without approach rumble strips, showed greater approach speeds compared to stop-controlled intersections at far distances, but lower speeds in close proximity (100 ft). This suggests that roundabouts are more effective at slowing drivers down near an intersection.

The roundabout geometry, particularly the splitter and central islands, is critical in limiting speed. Both islands must be designed to be conspicuous while preventing excessive sight distance, which encourages high speed (Ritchie & Lenters, 2005). Whereas roundabouts on low speed roads may have significant entry deflection, this design can result in crashes on the approach curve when applied on high-speed roads. Insufficient entry deflection encourages high entry speed and can shift accidents from the approach curve to the circulation. Hence, the splitter island entry deflection must be properly balanced, serving as a compromise between these two scenarios.

2.4 Heavy Vehicle Rollover

The rollover propensity of trucks on horizontal curves becomes an issue if drivers tend to drive excessively fast for the conditions. Highway on and off ramps that have tight curves are a well-recognized example, as studied by Green (2002) and McKnight and Bahouth (2009), the latter using data from the Large Truck Crash Causation Study (LTCCS). This

TABLE 2.4
Mean Speed Comparison of Roundabout and Stop-Controlled Approaches.

| Distance from Yield Line/Stop Bar (ft) | Roundabout (mph) | Stop-controlled (mph) | Difference (mph) |
|--|------------------|-----------------------|------------------|
| 100 | 26.4 | 28.9 | 2.5 |
| 250 | 35.5 | 34.8 | -0.7 |
| 500 | 45.3 | 45.0 | -0.3 |
| 1500 | 53.9 | 52.6 | -1.3 |

Source: Isebrands et al. (2014).

problem may be further elevated with a growing number of roundabouts built on high-speed roadways, despite the fact that the roundabouts can reduce overall crashes.

When cornering a tight curve such as that of a roundabout, small vehicles such as passenger cars tend to skid instead of roll (Harwood, Torbic, Richard, Glauz, & Elefteriadou, 2003). However, a rollover risk is introduced for long, heavy vehicles such as semi-trailers. Roundabout geometric features that are associated with an increased risk of rollover include: approaches with high speeds, small entry deflection, low-circulating traffic volume, excessive visibility, a significant decrease in radius within the roundabout, and sudden crossfall changes (Highways England, 2007). The first four factors are related to excessive speed on the approach and entry, while the latter two are associated with the road geometry.

Although the influence of the roundabout layout on overturning has been well studied, the effect of the circulating roadway superelevation is not well understood and has been suggested for further research (Gingrich & Waddell, 2008). Circulating speeds are known to be similar for inward vs. outward slopes (Gingrich & Waddell, 2008). This is important as it suggests that drivers do not discern these differences in superelevation. Differences do arise in the lateral force component experienced by a vehicle in these situations.

Vehicle factors relevant in truck overturning include speed, track width, center-of-gravity height, suspension, and tires (New Zealand Transport Agency, 2008). Furthermore, load factors such as overall weight and longitudinal and lateral weight distribution contribute to the rollover propensity (Harwood et al., 2003). Fully loaded semi-trailers tend to have a higher center of gravity height compared to those that are empty. A one-inch increase in the center of gravity height reduces the threshold necessary for initiating rollover by 0.005 G (Harwood et al., 2003).

Previously, systems for quantifying the proximity to rollover for tractor semitrailers have been introduced, including Winkler, Ervin, and Hagan (1999) who introduced a Roll Stability Advisor to determine the quasi-static threshold. Winkler (2000) and Gertsch and Eichelhard (2003) have also used controlled experiments, such as a tilt-table, to determine the rollover threshold of heavy vehicles. However, few studies have been done using field-collected data to determine how close heavy vehicles encroach on the critical rollover threshold, especially in the context of roundabouts on high-speed roads.

A quasi-static model provides a convenient means for considering rollover in road design (Gillespie, 1992) without requiring inputs difficult to attain in observational studies. In simplest form, the model involves the ratio (b/h), where b is half of the vehicle width and h is the height of the center of gravity. This ratio is sometimes referred to as the static stability factor or static rollover threshold. A version with roadway cross slope e that is more practical for design, was proposed

by Gillespie (1992) and later by Milliken and de Pont (2005):

$$a_r = (b/h - e)g \quad (2.1)$$

Where: a_r = critical lateral rollover acceleration
 b = half of the vehicle's width
 h = height of the center of gravity
 e = cross-slope or superelevation of the roadway
 g = acceleration due to gravity

Equation 2.1 is a 2D quasi-static approximation useful for considering road design matters if the following assumptions are met:

- The superelevation is uniformly applied along the curve,
- The driver follows the road curvature exactly,
- The vehicle body is parallel to the lane edge, and
- The vehicle moves at a constant speed.

These conditions are violated by heavy vehicles negotiating roundabouts. A roundabout paved surface may have complex elevation design, heavy vehicles such as semi-trailers follow unique paths that are different from the circulatory road alignment, tractors and trailers rarely stay parallel to the travelled way edge, and drivers adjust their speed along their paths frequently. These conditions need to be properly addressed in a 3D model that is more general and elaborate than Equation 2.1. Such a model is derived in Chapter 3.

2.5 Current Roundabout Capacity Models

Several models have been developed for roundabout capacity analysis. The most common approaches to modeling roundabouts include the empirical approach, gap-acceptance theory, and microscopic simulation. The empirical models are statistical and utilize regression to estimate the relationship between capacity and the geometric characteristics of roundabout (e.g., the UK Transport Research Laboratory (TRL) model).

The gap acceptance models are based on the mechanism of accepting or rejecting gaps in the major stream (circulating roadways on roundabouts) by drivers on the minor stream (approach roadways) (e.g., the Australian SIDRA INTERSECTION software model). The HCM 2010 capacity method includes a simple exponential regression model, in which the regression coefficients are based on gap acceptance behavior rather than the geometry of roundabouts. However, the method considers geometry in terms of the number of lanes. The simulation methods are computer-based programs that have the capability of simulating traffic and driver behavior at the microscopic level; VISSIM is one such software program. The concepts, main parameters, and limitations of each type of model are briefly discussed in the following sections.

2.5.1 UK Empirical Capacity Model

In the empirical method, the effort is concentrated on developing a mathematical relationship between the entry capacity and the circulating flow rate based on significant factors that may affect the relationship. This relationship is assumed to be linear or exponential, as shown in Equations 2.2 and 2.3 (Yap, Gibson, & Waterson, 2013). The coefficients are determined through statistical multivariate regression analysis.

$$q_e = A - B \cdot q_c \quad (2.2)$$

$$q_e = A \cdot \exp(B \cdot q_c) \quad (2.3)$$

Where: q_e = entry capacity (pc/h)
 q_c = circulating flow rate (pc/h)
 A and B are functions of the roundabout geometry

One well-known empirical model is the LR942 Linear Regression Model, which is most commonly used in the U.K. In this model, the entry capacity rate has a linear regression relationship to the circulating flow rate. The geometric characteristics of the entry roadways and the circulatory roadways are the main regression parameters. The model is shown in Equation 2.4 below.

$$Q_e = k \cdot (F - f_c \cdot Q_c) \text{ for } f_c \cdot Q_c \leq F, \text{ else } 0 \quad (2.4)$$

$$k = 1 - 0.00347(\varphi - 30) - 0.978(1/r - 0.05)$$

$$F = 303x_2$$

$$f_c = 0.21 T_D (1 + 0.2x_2)$$

$$T_D = 1 + \frac{0.5}{1 + \exp(\frac{D-60}{10})}$$

$$x_2 = v + (e - v)/(1 + 2S)$$

$$S = 1.6(e - v)/l'$$

Where: Q_e = maximum entry flow (veh/h)
 Q_c = circulating flow (veh/h)
 e = entry width (m)
 v = approach half-width (m)
 l' = effective flare length (m)
 r = entry radius (m)
 φ = entry angle ($^\circ$)
 S = measure of the degree of the flaring
 D = inscribed circle diameter (m)

The available software packages for the U.K. model are RODEL and ARCADY. Since the UK model is fully empirical and no theoretical basis exists to relate the capacity and the geometric characteristics, the model may not be applicable for U.S. roundabouts. According to the findings of NCHRP Report 572

(Rodegerdts, Blogg, et al., 2007), which is considered the largest body of research on U.S. roundabouts, the detailed geometric features as reflected in the U.K. model have no significant effect on the capacity of a roundabout; rather, the aggregate level in terms of the number of lanes is able to capture the geometric effects.

2.5.2 Gap-Acceptance Capacity Models

Gap-acceptance models are developed based on the availability of the headways in the major stream traffic (circulating traffic on roundabouts) and driver gap-acceptance behavior in terms of critical headway and follow-up headway. The Australian SIDRA INTERSECTION model and the HCM 2010 capacity model fall into this category. Although the SIDRA and HCM models are developed based on the same approach, their assumptions for arrival headway distribution (in circulating traffic for roundabouts) are different. The SIDRA model is developed based on a bunched exponential assumption while the HCM model is developed based on a simple exponential assumption (Akçelik, 2011; Rodegerdts, Blogg, et al., 2007). The SIDRA INTERSECTION model is shown in Equations 2.5 through 2.7.

$$Q_e = \max(Q_g, Q_m) \quad (2.5)$$

$$Q_g = \frac{3600}{t_f} [1 - A_m q_m + 0.5 t_f \phi_m q_m] e^{-\lambda(t_c - A_m)} \quad (2.6)$$

$$Q_m = \min(q_e, 60n_m) \quad (2.7)$$

Where: Q_e = maximum entry capacity (veh/h)

Q_g = gap-acceptance capacity (veh/h)

Q_m = minimum capacity (veh/h)

q_e = entry flow rate (veh/h)

q_m = arrival flow rate (veh/h)

n_m = minimum number of entry vehicles that can depart under heavy circulating flow conditions (veh/min)

λ = arrival headway distribution factor (veh/h)

$$\lambda = \frac{\phi_m q_m}{1 - \Delta_m q_m}$$

A_m = intra-bunch minimum headway in circulating traffic (sec)

ϕ_m = proportion of free (un-bunched) circulating vehicles

t_c = critical headway (sec)

t_f = follow-up headway (sec)

As can be seen in Equation 2.6, critical headway and follow-up headway are among the main parameters. Default values for these parameters have been incorporated into the model and computer-based programs such as SIDRA INTERSECTION software, which is based on Australian research and practice. As shown in

Table 2.6, the gap acceptance parameters for Australian drivers are considerably smaller than those of the U.S. If SIDRA standard software is used for capacity analysis of U.S. roundabouts without adjustment, an overestimation of the capacity can be expected. The NCHRP Report 572 findings also indicated that the aaSIDRA (2.0) model overestimates the capacity for U.S. roundabouts.

However, the assumptions of a congested condition (bunched) and a free condition (unbunched) for the arrival flow of a major stream (circulation) in SIDRA INTERSECTION appears to be reasonable for gap acceptance capacity models, and the traffic arrival pattern is not always expected to be random (Poisson). Therefore, evaluation of these assumptions for the HCM capacity model for U.S. roundabouts is recommended in the future.

2.5.3 HCM 2010 Capacity Model

Prior to 2000, limited research was performed on roundabouts in the U.S. because this type of intersection was not commonly used throughout the country. Deterministic software methods, such as RODEL, and simulation methods, such as VISSIM, based on U.K. and German research practice, respectively, have been used since 1990 (Rodegerdts, Bansen, et al., 2010). Chapter 17 of HCM 2000 provided a model for roundabout capacity analysis, but the model was restricted to single-lane roundabouts.

As roundabouts became increasingly popular, more studies were conducted on U.S. roundabouts. In 2007, NCHRP Report 572 presented the results of an in-depth investigation of the broad aspects of roundabouts, including safety, capacity, and design. In Chapter 4 of that report, a lane based exponential regression model was recommended for capacity analysis of single-lane and dual-lane roundabouts, as shown in Equations 2.8 through 2.10. It is worth mentioning that the capacity-related research findings of NCHRP Report 572 were incorporated in HCM 2010 in Chapter 21, a new chapter for roundabouts.

$$C_e = A e^{(-B v_c)} \quad (2.8)$$

$$A = \frac{3600}{t_f} \quad (2.9)$$

$$B = \frac{t_c - (t_f/2)}{3600} \quad (2.10)$$

Where: C_e = entry capacity (pc/h)

v_c = circulating flow rate (pc/h)

t_c = critical headway (sec)

t_f = follow-up headway (sec)

For single-lane roundabouts, the default values for A and B are 1,130 and 0.001, respectively. The same values are suggested for two entry lanes approaching one circulatory lane. For a single entry lane approaching

two circulatory lanes, the value of A is the same as for the single-lane while B is 0.0007. In addition, for roundabouts with two entry lanes approaching two circulatory lanes, the value of A is the same while B varies for different lanes: 0.00075 for a left lane and 0.0007 for a right lane. These differences are shown graphically in Figure 2.1. As can be seen in Equations 2.9 and 2.10, functions A and B depend upon the two main parameters, critical headway and follow-up headway. Therefore, it can be concluded that the accuracy of the HCM model depends on how well these parameters are estimated.

2.5.4 Simulation Methods

Simulation models are an alternative to empirical and analytical methods. These models are able to simulate traffic flow based on the car-following, lane-changing, and gap acceptance behaviors of drivers at intersections (Rodegerdts, Bansen, et al., 2010). Simulation software such as VISSIM is available for analyzing the capacity of individual intersections or intersections within a corridor/network. To analyze roundabout capacity in VISSIM, the default values for the gap acceptance parameters should be adjusted to reflect the behavior of local drivers.

2.6 Previous Studies on Gap-Acceptance Parameters

Many past studies estimated the two fundamental capacity parameters (critical headway and follow-up headway). A large research effort on roundabouts in the U.S. was conducted in NCHRP Project 3-65, the results of which were published in the NCHRP Report 572. The gap-acceptance parameters were estimated based on data from 18 approaches (roundabouts

located in urban/suburban areas) in five states. Table 2.5 shows the estimated parameters for single-lane and dual-lane roundabouts. These values were incorporated in the HCM 2010 capacity model for roundabouts. Moreover, many studies were conducted to estimate these values for individual states. Xu and Tian (2008) studied ten roundabouts in California and concluded that the estimated critical headways were consistent with the values reported in NCHRP 3-65 while the estimated follow-up headways were considerably smaller.

Previous research on roundabouts in Indiana also indicated that the critical headways and the follow-up headways were significantly lower compared to those presented in NCHRP Report 572. Tarko et al. (2008) studied a single-lane roundabout in Carmel, Indiana and estimated the mean critical gap as 3.1 sec and the average follow-up headway as 2.4 sec. Wei and Grenard (2012) also studied three single-lane roundabouts in Carmel to calibrate the HCM 2010 capacity model for single-lane roundabouts for local conditions. The study

TABLE 2.5
Summary of Critical and Follow-Up Headways for U.S. Roundabouts (Average Values in Parentheses).

| Field Measurements | Single-Lane | | Dual-Lane | |
|--------------------|------------------------|-------------------------|------------------------|-------------------------|
| | Critical Headway (sec) | Follow-up Headway (sec) | Critical Headway (sec) | Follow-up Headway (sec) |
| Approach | 4.2–5.9 (5.1) | 2.6–4.3 (3.2) | na | na |
| Right Lane | na | na | 3.4–4.9 (4.2) | 2.7–4.4 (3.1) |
| Left Lane | na | na | 4.2–5.5 (4.5) | 3.1–4.7 (3.4) |

Source: NCHRP Report 572 (Rodegerdts, Blogg, et al., 2007).
na = not applicable

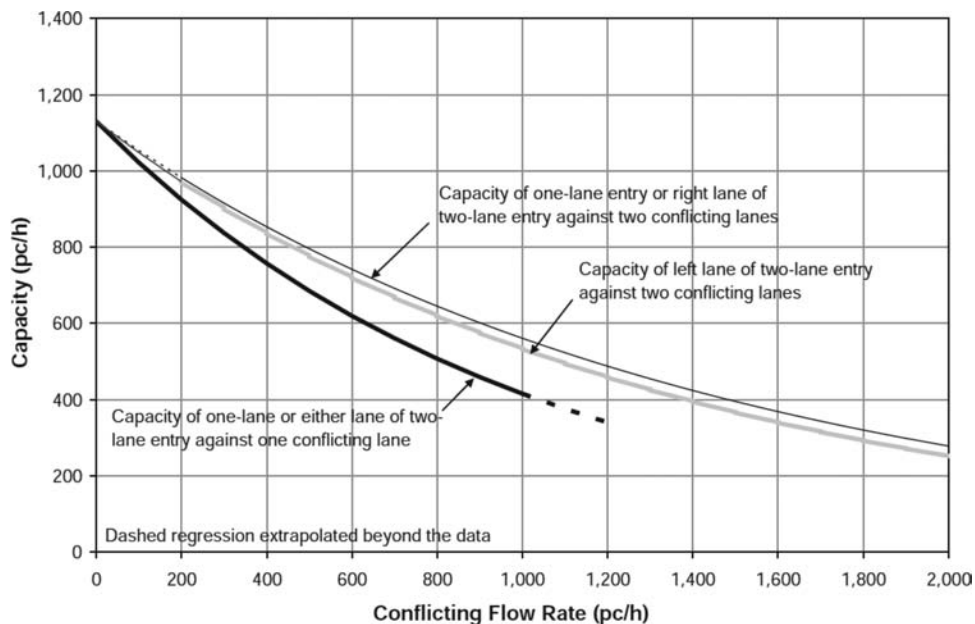


Figure 2.1 HCM 2010 lane-based capacity for roundabouts. (Source: HCM, 2010.)

estimated the average critical headway as 3.5 sec and the average follow-up headway as 2.2 sec. Day et al. (2013) collected a large amount of data from another single-lane roundabout in Carmel and measured the median critical gap as 2.2 sec. The aforementioned studies examined driver behavior on roundabouts on low-speed roads in the daytime with a low presence of heavy vehicles. Therefore, these findings are not transferable to larger roundabouts on state highways with a considerable presence of heavy vehicles.

Gap-acceptance parameters vary across countries. The estimated parameters for selected countries are shown in Table 2.6. The differences in gap-acceptance values indicate that the behaviors of drivers vary, which could be due to their roundabout driving experience and risk acceptance level. However, the lack of a standard methodology may affect estimation due to the initial assumptions, which will be discussed in Chapter 3. A proper methodology and accounting for the influencing factors would yield more accurate capacity estimations.

2.7 Factors Influencing Driver Gap-Acceptance Behavior

2.7.1 Heavy Vehicles

The presence of heavy vehicles is expected to reduce the capacity of roundabouts. Rodegerdts, Blogg, et al. (2007) reported that their parametric analysis for evaluating the correlation of heavy vehicles with the gap-acceptance parameters indicated a negative value, but the authors stated that this result was not confirmed and needs further exploration. On the other

TABLE 2.6
Gap-Acceptance Parameters for Selected Countries.

| Roundabout | Critical Headway (sec) | Follow-up Headway (sec) | Cited |
|-----------------------------|------------------------|-------------------------|----------------------------------|
| Australia | | | (Vasconcelos et al., 2013) |
| 1-Lane | 1.4-4.9 | 1.8-2.7 | |
| 2-Lane (Left) | 1.6-4.1 | 1.8-2.2 | |
| 2-Lane (Right) | — | 2.2-4.0 | |
| Germany | | | (Vasconcelos et al., 2013) |
| [1/2] 40 ≤ D ≤ 60 m | 5.6 | 2.5 | |
| [2/2] compact 40 ≤ D ≤ 60 m | 5.2 | 2.2 | |
| [2/2] large D > 60 m | 4.4 | 2.9 | |
| Turkey | | | (Tanyel, Baran, & Özuysal, 2007) |
| 1-Lane | 4.5-6.2 | 2.6-2.9 | |

[x/y]: Indicates number of entry lanes and circulatory lanes, respectively.

D: Inscribed Circle Diameter

hand, a study by Wisconsin DOT (2011) on four roundabouts (two single-lane and two dual-lane) located in Wisconsin indicated longer gap-acceptance parameters for trucks compared to passenger cars. The study reported the differences as 0.1 to 3.1 sec for critical headways and 0.2 to 1.4 sec for follow-up headways. Likewise, Dahl and Lee (2012) concluded that the critical headways and follow-up times for trucks were higher than for cars based on the data from 11 roundabouts located in Vermont, Wisconsin, and Ontario, Canada. In their study, the average critical headway was estimated as 4.3 sec for cars and 5.2 sec for trucks, indicating a 0.9 sec longer critical headway for trucks. Fitzpatrick, Abrams, Tang, and Knodler (2013) also estimated a longer critical headway for heavy vehicles compared to cars based on a single-lane roundabout located in Amherst, Massachusetts; the critical headways for cars and heavy vehicles were 2.2 sec and 2.8 sec, respectively, which indicate that heavy vehicles accept a 0.6 sec longer critical headway, on average, than cars.

Although a larger critical headway is expected for heavy vehicles, studying more cases will increase the body of knowledge regarding heavy vehicle gap-acceptance behavior on roundabouts built on high-speed roads.

HCM considers the effect of heavy vehicles on capacity in terms of an adjustment factor (i.e., converting heavy vehicle flow to passenger car equivalent (pce) as shown in Equations 2.11 and 2.12. According to HCM, the adjustment factor for trucks is 2.0. However, Lee (2014) concluded that trucks on a roundabout affect the capacity more than this adjustment. The adjustment factor was estimated as 3.0 for a circulating flow rate between 540–840 pce/h.

$$v_c = \frac{V}{f_{HV}} \quad (2.11)$$

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} \quad (2.12)$$

Where: v_c = circulating flow rate (pce/h)

V = demand flow rate (veh/h)

f_{HV} = heavy-vehicle adjustment factor

P_T = proportion of demand volume (at circulatory lanes) that consists of heavy vehicles

E_T = passenger car equivalent for heavy vehicles (the default HCM value for E_T is 2.0)

SIDRA Solutions (2012) suggested adjusting the gap-acceptance parameters rather than the flow rate with Equations 2.13 and 2.14. The heavy vehicle adjustment factor is to be calculated with Equation 2.12.

$$t'_c = \frac{t_c}{f_{HV}} \quad (2.13)$$

$$t'_f = \frac{t_f}{f_{HV}} \quad (2.14)$$

Where: t'_c = adjusted critical headway

t'_f = adjusted follow-up headway

f_{HV} = heavy-vehicle adjustment factor

On the other hand, a volume-weighted method for adjusting gap-acceptance parameters was introduced by Dahl and Lee (2012). According to this approach, the representative gap-acceptance parameters can be calculated from Equations 2.15 and 2.16. A separate analysis for estimating the gap-acceptance parameters for cars and trucks was recommended; and the adjusted gap-acceptance parameters using the above equations can be used as inputs to any gap-acceptance capacity models (Dahl & Lee, 2012). This approach appears to be a reasonable way to adjust gap-acceptance parameters to capture the effect of truck traffic on the entry capacity.

$$t'_c = t_{c,C} \cdot (1 - P_{TE}) + t_{c,T} \cdot P_{TE} \quad (2.15)$$

$$t'_f = t_{f,CC} (1 - P_{TE})^2 + (t_{f,CT} + t_{f,TC}) (1 - P_{TE}) P_{TE} + t_{f,TT} \cdot P_{TE}^2 \quad (2.16)$$

Where: t'_c = adjusted critical headway

t'_f = adjusted follow-up headway

P_{TE} = percentage of trucks at entry lanes

Sub C stands for car and sub T stands for truck (e.g., sub CT means car following truck), and all other terms are as defined previously.

Lee and Khan (2013) improved the volume-weighted approach by accounting for the truck traffic at both the entry and at the circulation roadways, as shown in Equations 2.17 and 2.18.

$$t_{c,C,i}' = t_{c,C,CC,i} (1 - P_{TC,i})^2 + (t_{c,C,CT,i} + t_{c,C,TC,i}) (1 - P_{TC,i}) P_{TC} + t_{c,C,TT,i} \cdot P_{TC,i}^2 \quad (2.17)$$

$$t'_{c,T,i} = t_{c,T,CC,i} (1 - P_{TC,i})^2 + (t_{c,T,CT,i} + t_{c,T,TC,i}) (1 - P_{TC,i}) P_{TC} + t_{c,T,TT,i} \cdot P_{TC,i}^2 \quad (2.18)$$

Where: $t'_{c,C,i}$ = denotes adjusted critical headway for cars approaching entry lane i

$t'_{c,T,i}$ = denotes adjusted critical headway for trucks approaching entry lane i

P_{TC} = percentage of trucks at circulatory lanes

Sub C stands for car and sub T stands for truck (e.g., sub CT means car accepting gap between a car and a truck), and all other terms are as defined previously.

The adjusted critical headways of cars and trucks based on the above equations are to be substituted with $t_{c,C}$ and $t_{c,T}$ of Equation 2.15, respectively. Although the suggested adjustments account for the possible effects of truck traffic on roundabout capacity, the estimation of several critical headways for different conditions which are less likely to happen (e.g., truck accepting a headway between two trucks on the circulation) may not be that desirable because such

details would require a relatively larger sample size to cover all the conditions.

2.7.2 Lighting Conditions

Limited research has been done on the effect of lighting conditions on the roundabout capacity. Tenekeci, Montgomery, and Wainaina (2010) studied several roundabouts in the UK in order to quantify the effects of adverse weather and lighting conditions on the entry capacity. In their study, data were collected utilizing video recording tools during different road surface and lighting conditions. The data were analyzed using the UK linear regression empirical model for roundabout capacity analysis; the results indicated that dry-dark conditions reduced the entry capacity by 6.3% on average for the entry saturation condition and 14.2% for the average circulation flow condition, which is comparable to the base condition of dry-light. The authors defined "dark" as a condition in which no natural light is present but rather is artificial. Burrow (1986) estimated a 5% reduction in roundabout capacity in the dark condition compared to the light condition (as cited in Tenekeci et al., 2010). Although their research quantified the impact of the dark condition on the entry capacity, the findings are not necessarily transferable to U.S. roundabouts. In addition, including the effects of the light condition on driver behavior is desirable for gap-acceptance capacity models.

2.7.3 Congestion

Driver behavior may be affected by the level of congestion on a roundabout as longer delays may lead to more aggressive actions. Congestion can be represented by control delay or the length of a queue on the approach. Delay also may be represented by the number of rejected gaps or waiting time at the first position of the queue. Mahmassani and Sheffi (1981) used a Probit procedure and data from actual observations to find the effects of delay on gap-acceptance behavior, represented by the number of rejected gaps, at an unsignalized intersection. They concluded that the critical headway is a decreasing function of the number of rejected gaps. Hamed, Easa, and Batayneh (1997) concluded that the waiting time at the first position of a queue at T-leg intersections affected driver behavior; the longer the waiting time was, the more likely the drivers were to accept shorter gaps. On the other hand, a study by Wisconsin DOT (2011) indicated that the effects of the queue length on the critical headways and follow-up headways were not significant.

The decision of the driver in the first position of a queue, who inspects the available headway, may be more critical than the other measures. In addition, a number of rejected gaps psychologically may determine the driver's decision more than the waiting time (i.e., by rejecting many gaps, the driver may think in terms of missed opportunities rather than the time delay). Also,

the queue length may not represent congestion well as a long queue can dissipate rather quickly if there is no or less circulating traffic, while a short queue will take longer time to dissipate if there is considerable circulating traffic. Therefore, the number of rejected headways, as a proxy, was considered to evaluate the effect of congestion on driver behavior.

On the other hand, generally, roundabouts on high-speed roads are less congested than those in urban areas, and only a few past studies therefore have addressed capacity-related driver behavior on such roundabouts. In order to have a better understanding of the operational performance of roundabouts on high-speed roads, it is important to know whether congestion affects driver behavior on the roundabouts located on those roads.

2.7.4 Other Factors

Road-surface condition (dry or wet) may affect driver behavior on roundabouts. A study by Tenekeci et al. (2010) on UK roundabouts indicated that the wet-light condition reduced the entry capacity by 7.1%, comparable to the dry-light condition. The weather effect on capacity-related driver behavior is not investigated in this report; however, it is important information for locations with extended rainfall seasons during the year. Therefore, it should be considered in future studies on roundabouts in the U.S.

2.8 Critical Headway Estimation Methods

Since the critical and follow-up headways strongly affect the capacity of a roundabout, valid estimation of these parameters is important. Various methods of gap-acceptance analysis are used for unsignalized intersections in general and for roundabouts in particular.

Raff's method, perhaps the oldest method for estimating critical gap, continues to be used in research. Fitzpatrick et al. (2013) used this method to estimate the critical headways for cars and trucks on a roundabout located in Amherst Massachusetts. Dahl and Lee (2012) also used this method for the same purpose on nine roundabouts in Wisconsin and Ontario, Canada, although they presented the estimated critical headways as the average of the Raff and Probability Equilibrium methods. Although Raff's concept is empirical and simple, Miller (1972) indicated that traffic volume variability affects critical headway estimation using this method (as cited in Brilon, Koenig, & Troutbeck, 1999).

The Probit method is another technique used for critical headway estimation. Daganzo (1981), Mahmassani and Sheffi (1981), and Hamed et al. (1997) used this method to estimate critical headways for unsignalized intersections, as well as the effects of other factors (e.g., waiting time and number of rejected gaps).

The Maximum Likelihood Method (MLM) has been widely used for estimating mean critical headways for

roundabout capacity analysis. Rodegerdts, Blogg, et al. (2007), Xu and Tian (2008), and Tarko et al. (2008) used this method to estimate the mean and standard deviation of critical headway on roundabouts.

The reliability of critical headway estimation methods have been evaluated in several studies. Brilon et al. (1999) described eight methods for critical gap estimation: the Sieglösch method for the saturated traffic condition and the lag, Raff, Harders, Logit, Probit, Hewitt, and MLM methods for unsaturated traffic conditions. The authors evaluated these methods with simulation for various generated traffic conditions for major and minor streams based on certain assumptions, and they concluded that the MLM and Hewitt methods produced the best results. The assumptions were shifted-Erlang distribution for critical and follow-up headways, hyper-Erlang distribution for traffic on major and minor streams, and consistent driver behavior (the driver maintains the generated critical headway until departure). However, generating major and minor traffic based on assumed distributions and consistent driver behavior degraded the robustness of the evaluation method. Therefore, the evaluation method could be improved with more realistic assumptions to reflect the actual traffic arrivals and to correspond to the assumptions of the estimation method in question (e.g., Probit assumes normal distribution for the critical headways, rather than shifted-Erlang distribution).

Tarko et al. (2008) performed a study to estimate driver gap acceptance parameters on roundabouts. Two methods of critical headway estimation were used in their study: the MLM and a new method that assumed inconsistent driver behavior (i.e., drivers may accept headways smaller than the earlier rejected ones). To evaluate the accuracy of the used methods, simulation was performed using VISSIM. The criterion for comparison was the service time in the first position of the queue. Based on a comparison of the service times of the simulated scenario and the actual one, it was concluded that MLM was preferred over the new method for the studied case. However, the comparison was based on the mean values only because the version of VISSIM they used did not allow entering the estimated standard deviations for the critical headway. It was suggested that the evaluation method could be improved by including both the mean and standard deviation of the critical headway in order to evaluate the assumption of driver consistency.

Vasconcelos, Luís, and Silva (2013) studied six roundabouts in Portugal and estimated their gap-acceptance parameters using the Raff, Wu (Probability Equilibrium Method), Troutbeck (MLM), Sieglösch, and Logit methods. The authors evaluated the accuracy of the methods by comparing the estimated (based on the estimated parameters) and the observed capacities (based on the field observations). Their general conclusion was that the estimated results were within the range of the observed capacities. Furthermore, it was implied that none of the methods were superior to the others.

Troutbeck (2014) used simulation to determine that the MLM can provide consistent and unbiased estimation of the mean critical gap while the Probability Equilibrium method could not.

Most of the past studies estimated the critical headways with the assumption that drivers are consistent (i.e., drivers always reject gaps shorter than the accepted ones); therefore, only the largest rejected gap and the accepted gap for each driver were considered in their analysis. This assumption can be questioned in light of research which indicated that some drivers reject gaps longer than the one they eventually accept, as was the case for the observations in this report.

Critical headway is a random variable that varies across drivers or even across the decisions of the same driver because of his/her different perception ability, risk acceptance, etc. Therefore, a certain distribution must be assumed and its parameters (mean and standard deviation) are the objective of estimation. Log-normal distribution has been assumed in many studies—particularly which used MLM, and is suggested by Troutbeck (2014). Wu (2012) concluded that the Weibull distribution better fitted critical headway, compared to the log-normal distribution. The conclusion was based on the Probability Equilibrium approach, which was introduced by the author (more details in Wu, 2012). In contrast, Troutbeck performed simulation and concluded that log-normal is preferred over Weibull distribution. Normal distribution was assumed in the past studies as it is the underlying distribution of the Probit method. However, it is implied that there is no strong empirical or theoretical basis to determine the distribution type of critical headways.

Although most of the above-mentioned methods have been used for estimation of critical headway, a tradeoff between the methods could be helpful. Therefore, the concepts, assumptions, and limitations of the widely used MLM and the Probit method are briefly discussed in the next section in order to select one of them as the preferred method for this report. In addition, simulation with more realistic assumptions will be helpful to verify the preferred method.

2.8.1 Tradeoff between the MLM and the Binary Probit Method

MLM is widely used for estimating the mean and standard deviation of critical headway. This method assumes that the driver's critical headway is between the largest rejected headway and the accepted headway and that the driver is consistent (i.e., always accepts a headway larger than the associated rejected headway). However, this method has the following limitations:

- For inconsistent driver behavior (i.e., the driver accepts a shorter gap than the largest associated rejected gap), the method recommends reassigning a value for the largest rejected gap just below the associated accepted gap (as

cited in Troutbeck, 2014). The data extraction in this report revealed that 5 to 10 percent of the observed drivers accepted shorter gaps than the largest associated rejected gap. Therefore, seeking alternative methods to account for this assumption may be desirable.

- The method assigns zero or a very small value for the absence of a rejected gap for drivers who accept the first gap (Troutbeck, 2014) because of its pairwise analysis approach. This assumption can also be questioned as this causes a biased sample due to the assumption of zeros for no rejected gaps.
- The method estimates the mean and variance of the critical headways only, as was used in NCHRP Report 572 and in Troutbeck (2014). The significance of explanatory variables other than the measured rejected or accepted headways were determined through a parametric analysis (Rodegerdts, Blogg, et al., 2007). A more convenient method would be to estimate the critical headway and determine the significance of the influencing factors.

On the other hand, the Probit method considers the driver's decision as a binary choice (i.e., the driver has the choice to reject or accept a gap). This method primarily could be preferred to the MLM for the following reasons.

- The assumption of driver inconsistency can be relaxed by including all the rejected headways and only the accepted headway for an individual driver, regardless which one is larger.
- There is no need to pair the headways (to assume values for the observations with no rejected headways) as this method considers rejection and acceptance decisions independent from one another.
- Typically, as many explanatory variables as available can be included in the model in order to determine their significance on the critical headway.

Another difference between these methods is the assumption of the critical headway distribution. MLM basically assumes a log-normal distribution where binary Probit assumes normal distribution. Troutbeck (2014) mentioned that log-normal is a reasonable distribution because of its non-negative property; however, the choice of other distributions was not rejected, because of the lack of strong empirical and theoretical bases. A problem with normal distribution can happen with a smaller mean and a larger standard deviation; in such a case, the probability of having negative critical headway values tends to increase. However, negative values could infer the condition of reversal priority (i.e., circulating traffic yields for entering traffic) in the case of heavy traffic on the approach.

As a result of the above discussions, the Probit method is primarily selected for estimating the critical headway parameters and determining the significance of the influencing factors on driver gap-acceptance behavior in this report. Besides, estimation results from MLM are also reported for comparison purposes. Furthermore, simulation is performed to validate the selected method.

2.9 Summary

Roundabouts on high-speed roads are emerging across the United States. The initial studies that have been conducted show crash reductions over traditional intersections, but the issue of heavy vehicle rollover has emerged as a safety concern for agencies such as state DOTs.

Equation 2.1 takes into account the roadway cross slope, or superelevation. However, it fails to account for variations in this superelevation. Furthermore, heavy vehicles such as semi-trailers follow complex paths that are different from the circulatory road alignment, and tractors and trailers rarely stay parallel to the roadway edge. The lateral tilt of the vehicle body in such cases can be quite different from the superelevation and is strongly influenced by the actual vehicle position. These issues need to be properly addressed by developing a rollover model more general than in Equation 2.1 that better reflects the complexity of the motion of long vehicles in a roundabout.

While studies have examined roundabouts in how they influence the speeds of approaching drivers, there have not been any studies comparing the heavy vehicle proximity to rollover for roundabouts on low and high-speed roads.

Furthermore, a key roundabout design parameter is the circulatory superelevation of the roadway. Outward superelevation is commonly used in the United States. Despite this, inward superelevation suggests a reduced rollover propensity (Gingrich & Waddell, 2008); however, the effect has not been quantified. An analysis is needed to determine whether the potential benefits afforded by inward superelevation design outweigh its shortcomings.

A subset of drivers are prone to aggressive behavior, which includes driving at excessive speeds. It is not known whether aggressiveness correlates with a higher rollover propensity at the roundabout. In the literature, this issue was recommended for further study to discern whether these drivers need special accommodation in the design process.

A factor that may affect rollover propensity relates to the “readability” of the roundabout, or how readily the driver is able to perceive the geometry. Drivers unfamiliar about how to properly traverse a roundabout can approach or decelerate too fast; as a result, their margin to rollover may be smaller than those whose behavior is more moderate. This warrants further analysis to determine if countermeasures are needed to offset this behavior.

Finally, given the prevalence of nighttime conditions during many northern latitude rush hour commutes, the effect of these conditions on the rollover threshold will also be examined.

3. RESEARCH METHOD

3.1 Safety Analysis

3.1.1 Background

The rollover scenario is generated by inertial forces acting around a vehicle’s rolling axis. These forces produce torques about the axis; the rollover tendency

comes primarily from the torque generated by centrifugal force, which passes through the vehicle center of gravity. Its magnitude is determined by the longitudinal speed and instantaneous curvature of the vehicle’s center of gravity path.

When the moment arm between the rolling axis and a force increases, the force can generate a larger torque. Thus, heavy vehicles with high centers of gravity tend to have a greater rollover propensity. When the vehicle is cornering, it will reach a speed at which rollover becomes imminent. This condition is called the critical speed and can be assessed by Δv , or the difference between the critical rollover speed and the actual vehicle speed at that moment. The quantity changes along the vehicle path and typically becomes smallest in the sharpest portion of the curve.

3.1.2 General Equation for Heavy Vehicle Rollover

In a simplified, two-dimensional model representing “quasi-static” rollover, the rolling axis can be considered as passing through the center of the footprint of the outside front and rear tires. Overturning occurs when the torque generated by the centrifugal force about the rolling axis is greater than that produced by the vehicle weight. This model assumes constant superelevation and can be derived from a free-body diagram (Figure 3.1). The normal force on the inside tires reaches zero just as the truck begins to tip.

Taking moments about point “A” (counterclockwise positive), the following expression is obtained:

$$-\frac{hmv^2}{r} \cos \theta - \frac{bmv^2}{r} \sin \theta - hmg \sin \theta + bmg \cos \theta = 0 \quad (3.1)$$

Where: v = speed of vehicle
 m = mass of vehicle
 r = radius of center of gravity path
 b = half the width between tires
 h = center of gravity height

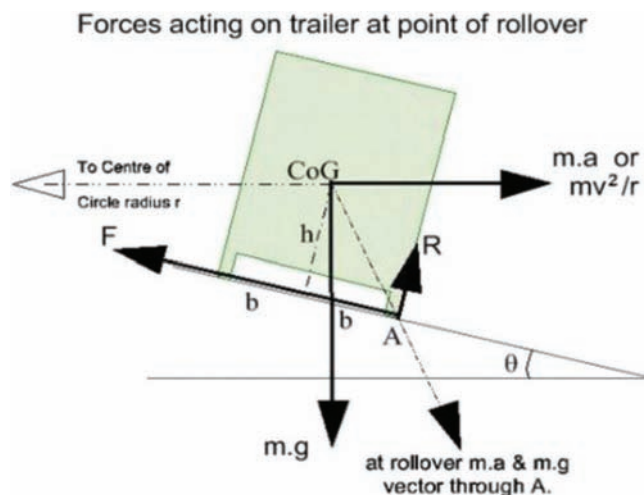


Figure 3.1 Components of the quasi-static rollover condition (Sawers, 2011).

g = acceleration due to gravity
 θ = superelevation of roadway

Rearranging, Equation 3.1 becomes:

$$v_{crit} = \sqrt{\frac{rg(bc\cos\theta - h\sin\theta)}{bsin\theta + hc\cos\theta}} \quad (3.2)$$

In this equation v_{crit} represents the critical speed at which rollover is initiated, the model does not account for changes in the cross slope. Heavy vehicles, such as semi-trailers, are often similar in size to the roundabout dimensions; hence, the path and corresponding elevation of points on the vehicle may be very different from one another. A more generalized model that accounts for the complexities of the actual vehicle position is needed.

A great diversity of models are used to assess the situation. Not only can vehicle factors be accounted for, but also pavement conditions and dynamic components such as suspension and tires. A considerable number of vehicles are analyzed in this analysis; hence, a three-dimensional static analysis provides a suitable approximation. The derivation of such a model is discussed for semi-trailers, the heavy vehicle type that is most prone to rollover.

3.1.3 Derivation of Rollover Model

The original derivation of the rollover model is from an unpublished research note (Tarko, Hall, & Lizarazo, 2014). The derivations below further refine these ideas and posit a new method for determining the critical rollover threshold.

Figure 3.2 presents a semi-trailer with the key elements of the model marked. The two rectangular areas beneath the wheels represent the tires' footprints. Points A and B are the midpoints of the outer sides of the footprint areas. Point E is the center of the so-called "fifth wheel" which facilitates a secure hinge between

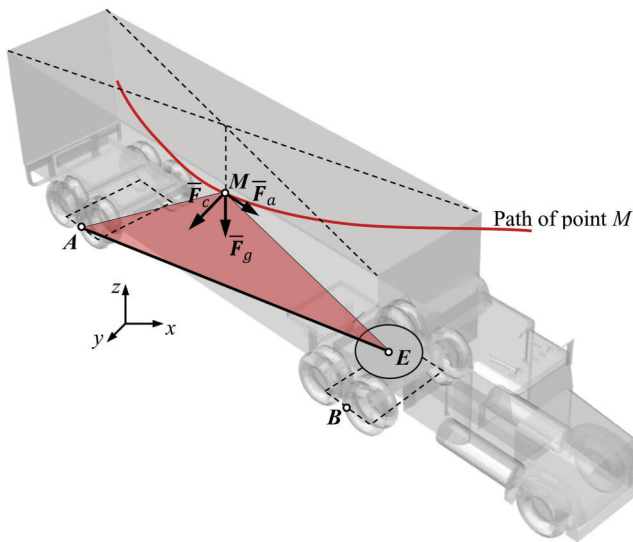


Figure 3.2 Key elements of the new 3D model of semi-trailer rollover.

the trailer and the tractor. The fifth wheel supports the front of the trailer. If the pavement surface and the dimensions of the trailer are known, then points A and B determine the position of the trailer in the system of coordinates (x, y, z) together with the positions of the other wheels and point E . The position of the trailer's center of mass M can also be determined if the load and its distribution in the trailer are known.

During a rollover, the trailer body rotates outside of the curve around an axis passing two points of support: point A and point E . The rollover is caused by a force \bar{F} composed of three pseudo forces: centrifugal \bar{F}_c , longitudinal \bar{F}_a , and gravity \bar{F}_g . The \bar{F} force can be decomposed into two components: (1) component $\bar{F}_n = \bar{n}_1 \bar{F}$ normal to the plane, and (2) component $(\bar{F} - \bar{F}_n)$. The unit vector is normal to plane AEM and can be calculated as the vector product: $(\overline{AE} \times \overline{AM}) / \overline{AE} \times \overline{AM}$ normalized with its length. Force \bar{F}_n is directly responsible for a rollover. If the \bar{F}_n force points inside the curve, then the torque produced by this force is counterbalanced by the pavement reaction to the pressing force of the inside wheels. On the other hand, if the \bar{F}_n force points outside of the curve, then there are no other forces that could counterbalance the torque produced by force \bar{F}_n and the trailer loses its lateral stability. This rollover condition can be expressed as:

$$\bar{n}_1 \cdot (\bar{F}_a + \bar{F}_c + \bar{F}_g) > 0 \quad (3.3)$$

The three forces can be calculated as follows:

$$\begin{aligned} \bar{F}_a &= ma \bar{u}_a \\ \bar{F}_c &= mc \bar{u}_c \\ \bar{F}_g &= mg \bar{u}_g \end{aligned} \quad (3.4)$$

Where: m = trailer mass

a = longitudinal acceleration

c = centrifugal acceleration

g = gravity acceleration

\bar{u}_a = unit vector tangent pointing towards the direction of movement

\bar{u}_c = unit vector pointing outside of the path's local curve

\bar{u}_g = unit vector pointing downward

To determine the critical rollover speed at which the trailer begins to tip, the inequality expression in Equation 3.5 is replaced with Equation 3.5.

$$\bar{n}_1 \cdot (ma \bar{u}_a + mc \bar{u}_c + mg \bar{u}_g) = 0 \quad (3.5)$$

Noting that $c = v^2 \rho$, where v is the longitudinal speed of the vehicle and ρ is the local curvature of the trailer's path, the relationship is rearranged to find the critical rollover speed v_{cr} :

$$v_{cr} = \sqrt{\frac{\bar{n}_1 \cdot (-g\bar{u}_g - a\bar{u}_a)}{\bar{n}_1 \cdot (\rho\bar{u}_c)}} \quad (3.6)$$

Finally, the critical speed v_{cr} can be compared to the actual speed v . This leads to an important surrogate measure of safety that indicates the proximity of rollover Δv :

$$\Delta v = v_{cr} - v \quad (3.7)$$

The values of Δv estimated at time points t along the vehicle's trajectory were used to study the trucks' propensity for rollover. The estimation Δv with the proposed model requires the trajectory of the studied vehicles, their dimensions, and the load information.

3.2 Capacity Analysis

3.2.1 General Approach

Following the widely accepted approach to capacity analysis, gap-acceptance data were analyzed and the critical and follow-up headways were estimated. Traffic operations on four roundabouts built on Indiana state highways were video-recorded with high-resolution cameras during the morning and afternoon peak hours during fall 2013 and spring 2014. Utilizing a special image analysis tool, developed at the CRS, headways were measured and other explanatory variables (shown in Table 4.5) were noted. The binary Probit concept was used for the estimation of the mean and standard

deviation of the critical headways, as well as for the evaluation of the influencing factors. The measured follow-up headways for each condition were averaged and the standard deviations were calculated.

In addition to a reasonable estimation technique, the gap-acceptance analysis in this report required proper preparation of data. NCHRP Report 572 considered three approaches for determining the inclusion of observations: (1) all accepted and rejected gaps and accepted lags, (2) observations that contained at least a rejected gap, and (3) observations where queuing was observed during the entire minute and contained a rejected gap. Method (2) was preferred in the NCHRP study. The concept of gaps and lags are shown in Figure 3.3.

Since a lag is a portion of a gap, inclusion of lags in the data set leads to underestimation of the critical headway (Rodegerdts, Blogg, et al., 2007; Tarko et al., 2008). Likewise, due to low to medium traffic volumes on the studied roundabouts, especially in rural areas, obtaining enough observations from a queue during the entire minute was not feasible. Therefore, the data set in this report followed method (2), which is consistent with the NCHRP Report 572 methodology.

Furthermore, it was assumed that driver behavior is inconsistent. Considering this assumption, each rejected headway contributes information about driver consistency. Information from the extracted observations in

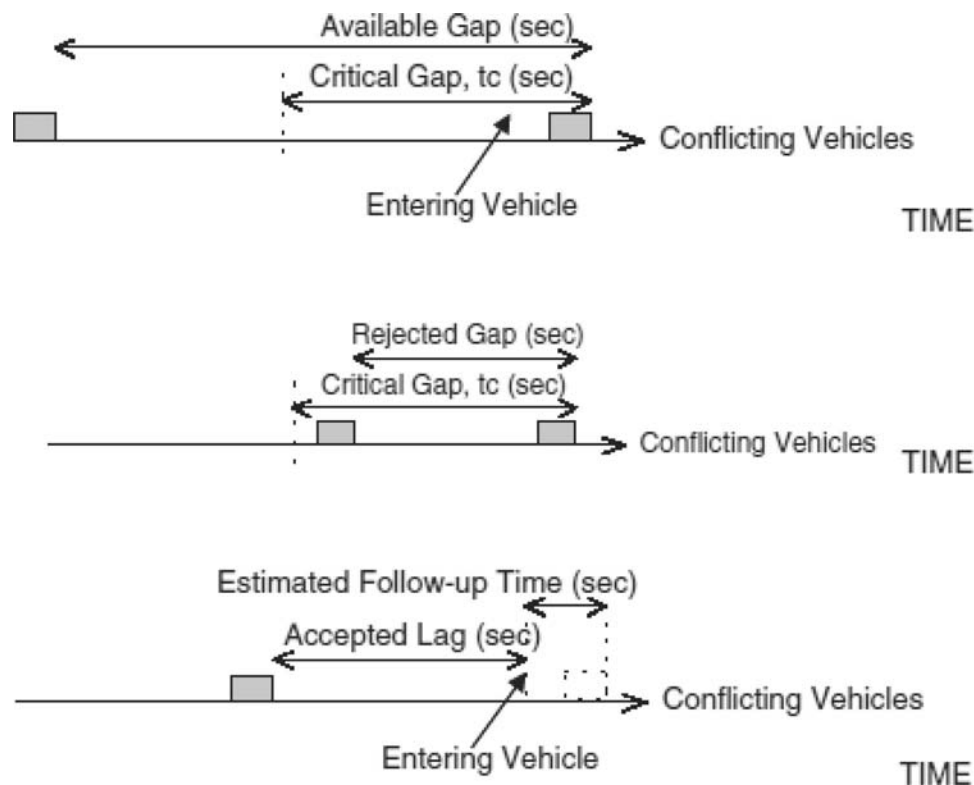


Figure 3.3 Concepts of gaps and lags. (Source: NCHRP Report 572 [Rodegerdts, Blogg, et al., 2007].)

this report confirmed inconsistent driver behavior as 5 to 10 percent of the drivers accepted shorter headways over the associated rejected headway(s). Therefore, all rejected headways and accepted headways were included in the analysis.

Unlike the MLM pairwise analysis, the Probit method considers each event (rejected or accepted) as an independent decision, even for the same driver. Therefore, all rejected and accepted headways were included for the model estimation, without any adjustment, as discussed earlier.

To evaluate the effects of the influencing factors on driver behavior and, in turn, on the capacity, a driver of a passenger car approaching a single-lane roundabout on a low-speed road during daylight conditions was set as the base case.

Finally, the assumptions and the techniques used for estimating critical headways were evaluated with simulation. The difference in the average delays (sec/veh) at the first position of the queue between the simulated scenarios and the actual observations was considered as the performance measure.

3.2.2 Binary Probit Method

The binary Probit concept was selected to estimate the critical headway and the effects of the studied variables. Let us assume that t_i is the shortest headway acceptable to a driver at the moment the driver inspects headway h_i . This shortest acceptable headway (critical headway) depends on some variables taking values X_i and other unknown conditions represented by error term ε_i at the time when headway h_i is inspected. The error term is assumed normally distributed with zero mean and standard deviation σ . Hence, the critical headway can be represented as Equation 3.8.

$$t_i = \beta X_i + \varepsilon_i \quad (3.8)$$

The probability of headway acceptance can be related to the duration of the available headway. The probability P that headway h_i is accepted is shown in Equation 3.9. Substituting t_i with its function results in a standard binary Probit model, as shown in Equations 3.10 through 3.12.

$$P(Y_i = 1 | X_i) = P(t_i \leq h_i) \quad (3.9)$$

$$P(Y_i = 1 | X_i) = P\left(\frac{\varepsilon_i}{\sigma} \leq \frac{h_i - \beta X_i}{\sigma}\right) \quad (3.10)$$

$$P(Y_i = 1 | X_i) = \Phi(\beta_h^* h_i - \beta^* X_i) \quad (3.11)$$

$$P(Y_i = 1 | X_i) = \Phi\left(\frac{1}{\sigma} h_i - \frac{\beta}{\sigma} X_i\right) \quad (3.12)$$

Where: Y = binary variable taking value 1 when headway is acceptable and value 0 otherwise

P = probability that headway accepted by a driver
 h_i = measured headway

Φ = the standardized cumulative normal distribution
 t_i = critical headway
 σ = standard deviation of a critical headway (the scaling parameter)

β_h = estimated parameter for the headway variable
 $\beta = (\beta_0, \beta_1, \beta_2 \dots)$ = estimable parameter for an intercept and other variables

$X = (1, X_1, X_2 \dots)$ = explanatory variable

Statistical Analysis Software (SAS) using the maximum-likelihood estimator was utilized to estimate the model parameters β^* in Equation 3.11. Then, the critical headway parameters—mean (μ) and standard deviation (σ)—were calculated from Equations 3.13 and 3.14, as reported by SAS Institute Inc. (2011).

$$\mu = \frac{\beta^*}{\beta_h^*} \quad (3.13)$$

$$\sigma = \frac{1}{\beta_h^*} \quad (3.14)$$

The t -statistic was used to determine the significance of the model coefficients. The significance level of 0.05 (95% confidence level) was used. The effects of the significant variables on roundabout capacity were evaluated by calibrating the HCM 2010 capacity model to reflect the local conditions.

3.2.3 Maximum Likelihood Method (MLM)

MLM was also used in the current research to estimate the mean and variance of the critical headways in order to ensure that the differences between the values estimated in this report and those of the NCHRP Report 572 were not due to different applied methodologies. The recommended procedure by Troutbeck (2014) was followed for the MLM, as described below.

If $F(a_i)$ and $F(r_i)$ are the cumulative distribution functions (cdf) of the accepted gaps and rejected gaps, respectively, then the likelihood (L) of the critical headway for an individual driver is:

$$L = F(a_i) - F(r_i) \quad (3.15)$$

The likelihood for the entire population of drivers is the product of the individual likelihoods as:

$$L = \prod_{i=1}^n [F(a_i) - F(r_i)] \quad (3.16)$$

The log-likelihood (LL) function is used for simplification as:

$$LL = \sum_{i=1}^n \ln[F(a_i) - F(r_i)] \quad (3.17)$$

To estimate the mean and variance of the critical headways, the log-likelihood function was maximized. An iterative process was required to maximize this function; a spreadsheet was utilized for this purpose. In this procedure, the initial values for the mean (m) and variance (s^2) were required as inputs. Log-normal distribution was assumed for the distribution of critical

headways. Eventually, the desirable parameters, the mean (μ), and the variance (σ^2) of the critical headways were calculated from Equations 3.18 and 3.19.

$$\sigma^2 = \ln\left(\frac{s^2}{m^2} + 1\right) \quad (3.18)$$

$$\mu = \ln(m) - 0.5\sigma^2 \quad (3.19)$$

3.2.4 Simulation

The assumptions and methods used in this report for estimating critical headways were evaluated with simulation. The assumptions for the Probit method were as follows: inconsistent driver behavior (may accept headways smaller than the earlier rejected ones) and normal distribution of critical headways across drivers; and the assumptions for the MLM were as follows: consistent driver behavior (always accept headways larger than the earlier rejected one) and log-normal distribution of critical headways across drivers. Based on the estimated models for critical headways, two possible scenarios were evaluated.

1. Inconsistent driver behavior and normal distribution of the critical headways. This scenario was evaluated based on the results from the Probit model, in which all rejected headways and accepted headways were included.
2. Consistent driver behavior and log-normal distribution of critical headways. As the Probit method is restricted to the normal distribution assumption, this scenario was evaluated based on the results from the MLM, in which the accepted headway and the largest rejected headway for the driver in question were included (with adjustment of the largest rejected headway just below the accepted headway in the case of a higher value).

The performance measure considered in the evaluation was the difference in the actual average delay (sec/veh) at the first position of the queue and that of the simulated scenarios since delay is one of the most important elements of the capacity analysis. To measure the actual time that the drivers spent in the first position of the queue, the Traffic Tracker tool, developed by CRS, was used to mark the real time for each driver maneuvering on the single-lane roundabout

for three hours. The information from the recorded time stamped was used to measure the individual observed delays.

In addition, the gap-acceptance parameters were estimated from the same observations. Then the gap-acceptance behaviors of the same drivers were simulated based on the estimated mean and standard deviation of the critical and follow-up headways. Finally, the actual delays from observations and those of the simulations were compared.

4. DATA

4.1 Data Collection

Roundabouts have been built on Indiana's state highway system since 2008. Given their location on state roads, a number of these roundabouts have approaches that are high speed (45 mph or greater).

It was desired to select nearby roundabouts: one on a high-speed road and the other on a low-speed road to discern the differences between these conditions while maintaining similar driver characteristics. Roundabouts were chosen in two areas: Lafayette and Noblesville. An additional roundabout in Valparaiso was also examined for the capacity analysis. Table 4.1 provides a description of the selected roundabouts.

The State Road (SR) 25 and Concord Road/Maple Point Drive roundabouts are located in Lafayette. The latter is a single lane roundabout and is not built on the state highway system. It was selected due to it being the only such low-speed roundabout with significant heavy vehicle traffic in the Lafayette area. Figure 4.1 and Figure 4.2 show the SR 25 and Concord Road/Maple Point Drive roundabouts, respectively.

The SR 32-38 roundabouts at Promise Road and Union Chapel Road are located on the edge of Noblesville on this main thoroughfare to nearby Anderson. Two of the approaches to the roundabouts are high-speed. The short connecting road in between has two low-speed approaches. The SR 32-38 roundabouts are shown in Figure 4.3.

For the capacity analysis, driver behavior was also studied on a low-speed approach roundabout on SR 130, located in an urban area of Valparaiso, Indiana.

TABLE 4.1
Description of Study Roundabouts.

| Roundabout | Number of Approaches | Highest Approach Speed | Number of Entry Lanes | Number of Circulatory Lanes and Width | Super-elevation | Approach Curve Radius | Inner Radius | Year Built |
|-------------------------------|----------------------|------------------------|-----------------------|---------------------------------------|-----------------|-----------------------|--------------|------------|
| SR 25 | 3 | 55 mph | 2 | 2 × 16 ft | -2% to 2% | 121 ft | 56 ft | 2012 |
| Concord Rd/Maple Point Dr | 3 | 30 mph | 1 | 1 × 16 ft | 2% | 90 to 93 ft | 60.5 ft | 2012 |
| SR 32-38/Promise Rd | 4 | 30 to 55 mph | 2 | 2 × 16 ft | 2% (varies) | 100 ft | 58 ft | 2011 |
| SR 32-38/Union Chapel Rd | 3 | 30 to 55 mph | 2 | 2 × 16 ft | 2% (varies) | 100 ft | 58 ft | 2011 |
| SR 130/ Laporte Ave/Sturdy Rd | 4 | 35 mph | Varies | 1 × 20 ft | Varies | 50 to 78 ft | 37 ft | 2008 |



Figure 4.1 SR 25 roundabout in Lafayette (Google Earth).

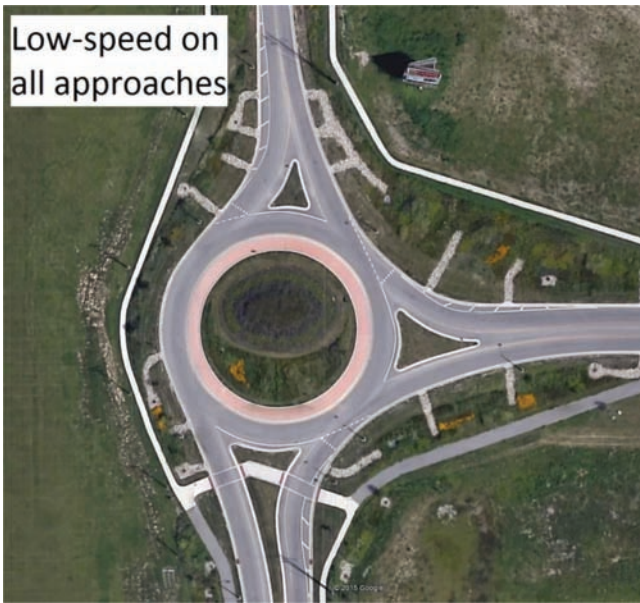


Figure 4.2 Concord Road/Maple Point Drive roundabout in Lafayette (Google Earth).

The speed limit for all the approaches of this roundabout was 35 mph. This roundabout is classified as a single lane, and its geometric configuration is shown in Figure 4.4.

All studied roundabouts were lighted in the circulation and each approach had signs upstream warning drivers of the approaching roundabout.

Well over one-hundred hours of video data were collected from the roundabouts. Data extraction was performed utilizing a special video tracking software developed in the Purdue Center for Road Safety. A summary of the heavy vehicles extracted is included in Table 4.2.

Due to the small sample of observations at the SR 32-38/Union Chapel Road roundabout in Noblesville, this portion of the data was not utilized in the safety analysis.

Data collection was facilitated by the Purdue Mobile Traffic Lab (MTL), a van featuring two high-resolution dome cameras mounted atop a 42 foot extendable mast. The data could be reviewed on the monitors in the back of the van and 4 terabytes of capacity were available for video storage. The van setup can be seen in Figure 4.5.

4.2 Data Extraction for Safety Analysis

Customized Vehicle Tracking Software (VTS) was developed to collect the point data along the vehicles' trajectories at a pre-specified time interval. The VTS user interface is shown in Figure 4.6. VTS displays video frames at the set frequency and an observer marks selected points of the vehicle's body. VTS records the monitor coordinates (x, y) of the selected points and adds time stamp t. In the post-processing phase, VTS converts the monitor-based (x, y, t) coordinates into the real-world 3D coordinates using a double-homology procedure and at least four reference points known in both the coordinate systems.



Figure 4.3 SR 32-38 roundabouts in Noblesville (Google Earth).



Figure 4.4 SR 130 roundabout in Valparaiso, IN (Google Earth).

TABLE 4.2
Heavy Vehicle Types Extracted for Safety Analysis at Each Roundabout and Approach.

| Roundabout Approach | Semi-trailers |
|---|---------------|
| SR 25 high speed (55 mph) | 116 |
| Concord Rd./Maple Point Dr. low speed (35 mph) southbound through | 34 |
| Concord Rd./Maple Point Dr. low speed (35 mph) southbound left | 26 |
| SR 32-38/Union Chapel Rd. high speed (55 mph) | 9 |
| SR 32-38/Union Chapel Rd. low speed (30 mph) | 10 |
| Total | 195 |

The use of two consecutive homological transformations eliminated finding the parameters of the mathematical projection formula (García & Romero, 2009).

The points marked at the vehicle's tires were used to extract the trajectories. The data extracted from video frames formed a sequence of location data points that

approximately represented the vehicles' trajectories (x,y,t) . Figure 4.7 shows points being marked along a vehicle path.

Marking the vehicle's location points on a video frame is manual; thus, the collected data are susceptible to error. This error is also present in the location data transformed to the real-world coordinates (x,y,t) . Therefore, estimation of the actual trajectories from the collected data is necessary to reduce the measurement error. To simplify this estimation, the errors are assumed approximately normal and identically and independently distributed.

Bezier curves are used to approximate the vehicle trajectory. The estimation task is to find a Bezier curve that well represents the trajectory of a vehicle whose motion is known via a sequence of location points over time. The least-squares technique is applied to the positions of estimated and measured points using a low-degree Bezier curve; this curve is repeated to a limited number of points while moving along the considered

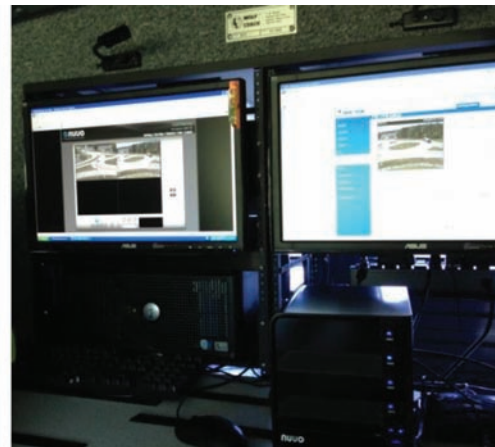


Figure 4.5 Purdue Mobile Traffic Lab (MTL) setup.

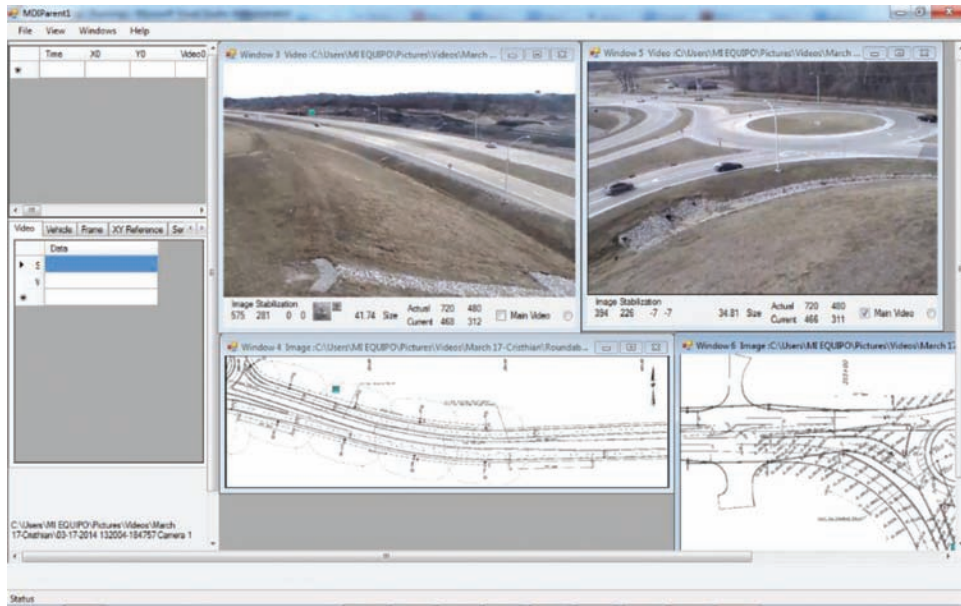


Figure 4.6 User interface “Vehicle Tracking Software” for data extraction.

road section with additional constraints to guarantee continuity and smoothness of the path and the vehicle’s motion in time.

We interpret the Bezier curve parameter τ as a time-related quantity. More specifically, it is the proportion of travel time that has elapsed since entering the road section considered. Not only does this approach estimate the vehicle’s position in space, but also the vehicle’s advancement along the road in time. Bezier curves allow one to obtain a smooth vehicle trajectory. The dynamic characteristics of the vehicle’s motion,

such as speed and acceleration, were calculated from the estimated trajectory.

Positions for each tire were estimated by including conditions that guarantee constant vehicle dimensions. Then, the z coordinate is calculated for each tire location based on a Delaunay triangulation road surface model. The location of the center of gravity and the rolling axis are calculated based on the dimensions of the trailer and the 3D positions of the trailer tires at time t. The positions of the trailer tires can be measured in the field together with the vehicle



Figure 4.7 Video tracking software.

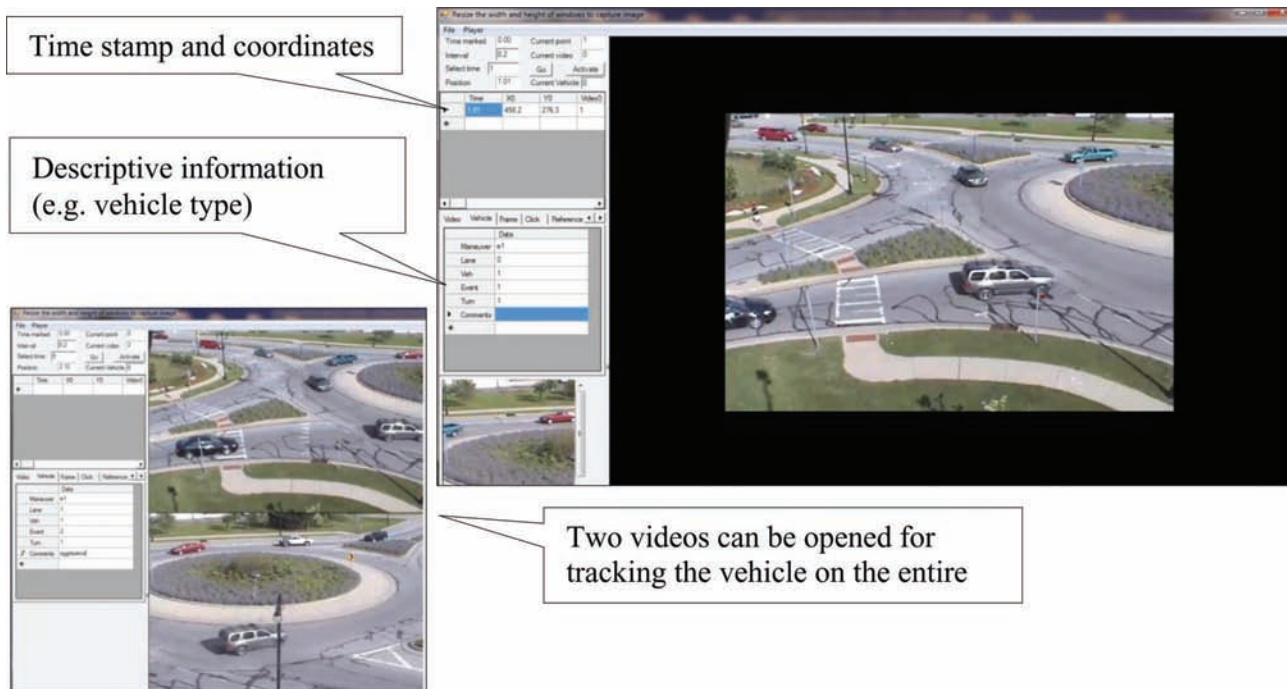


Figure 4.8 Screen shot of the data extraction tool.

speed. Thus, the general rollover condition can be applied to the actual or designed pavement elevation and to the actual behavior of truckers on the road.

Since the weight distribution of the vehicles was unknown, two distinct cases may be assumed for study vehicles: unloaded and loaded. In this study, for the unloaded case, a standard-sized trailer weighing approximately 12,640 lb was considered. For loaded trailers, studied vehicles were assumed to be at the federal maximum gross vehicle weight: 80,000 lb in the United States (FHWA, 2003), with the load evenly distributed and filling the vehicle to half of its capacity. While the actual vehicle weight is expected to be somewhere in between, the unloaded and loaded cases provide upper and lower bounds of the rollover threshold Δv .

The rollover model is applicable to various types of roadway curves if the vehicles' trajectories (vehicles' positions at each point in a time series) along these curves are known.

4.3 Data Extraction for Capacity Analysis

The rejected/accepted and follow-up time headways were extracted with a special image analysis tool developed by CRS. This tool has the ability to record time stamps in one-tenth of a second as well as the local coordinates. Other information about the roundabouts (e.g., lane use, turning movement, vehicle type, weather conditions, visibility conditions, and aggregate geometric characteristics (number of lanes) also was noted. A screen shot from the tool is shown in Figure 4.8. During the data extraction from two-lane roundabouts

(dual circulatory lanes), it was observed that entering vehicles yielded, to all the circulating vehicles, regardless of the lanes.

For measuring the observed headways, the following definitions were helpful and are graphically illustrated in Figure 4.9 and Figure 4.10; however, engineering judgment was also valuable.

Yield line: the outer edge of the circulatory lane (outer lane in multiple-lane roundabouts) within an approach. This line is not always the marked yield line.

Conflict line: the left edge of a corridor used by a vehicle entering the circulatory lane from an approach.

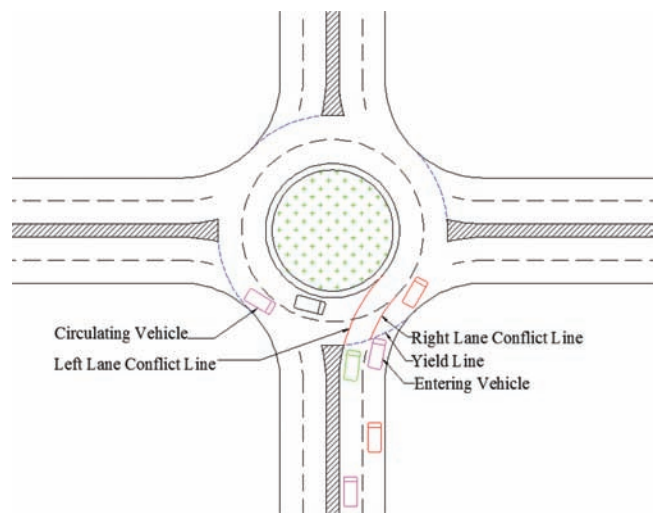


Figure 4.9 Vehicles interaction and conflict area.

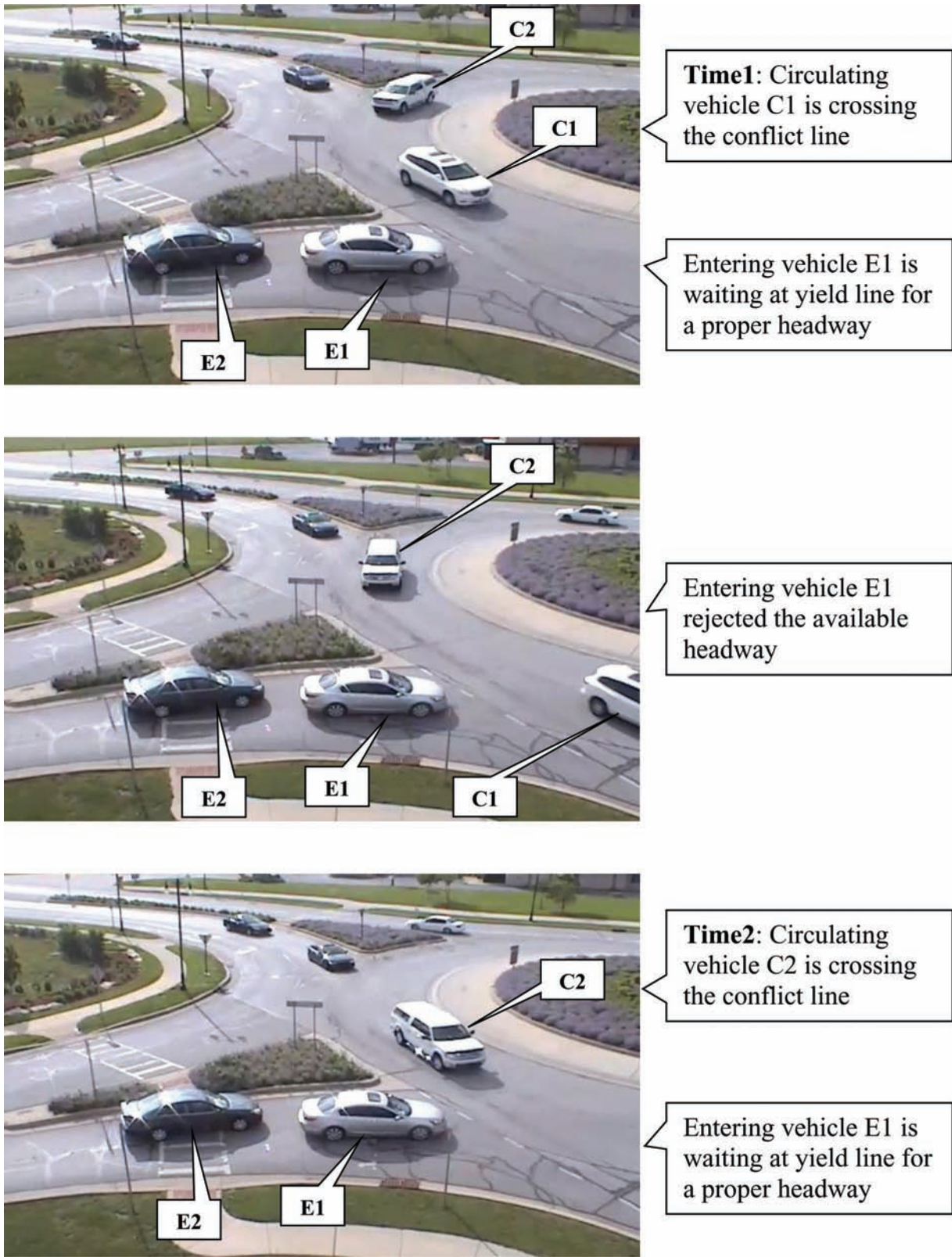
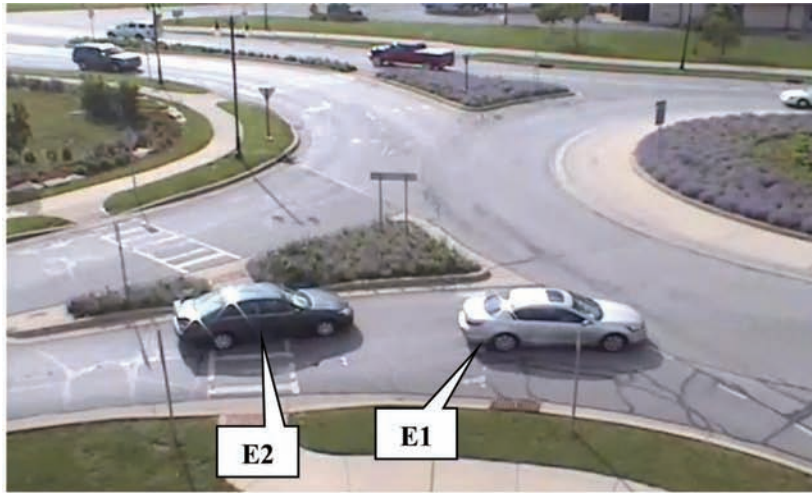


Figure 4.10 Illustration of rejected, accepted, and follow-up headways. (Continued on next page.)



Time3: Entering vehicle E1 is crossing the yield line (accept the headway)



Time4: Entering vehicle E2 is crossing the yield line (accept the same headway as E1 did)



Time5: Circulating vehicle Cn is crossing the conflict line

Entering vehicle En is approaching the yield line

Figure 4.10 Illustration of rejected, accepted, and follow-up headways. (Continued.)

TABLE 4.3
Sample Size and Date of Data Collection.

| Roundabout | Approach | Sample Size | | Date of Data Collection |
|---|----------|---------------------------|-------------------|-------------------------|
| | | Rejected/Accepted Headway | Follow-up Headway | |
| SR 25: Old SR 25, Lafayette | E | 160 | 47 | October 2013 |
| SR 32/38: Union Chapel Road, Noblesville | N | 365 | 130 | May 2014 |
| SR 32/38: Promise Road, Noblesville | S | 181 | 30 | December 2013 |
| Indiana 130: LaPorte Ave–N. Sturdy Road, Valparaiso | All | 2,193 | 606 | June 2014 |

Entering vehicle: a vehicle passing with its front bumper at the yield line and continuing into the roundabout.

Circulating vehicle: a circulating vehicle that crosses the conflict line. A circulating vehicle in any of the two circulatory lanes is circulating for a vehicle entering the roundabout from the left approach lane. A circulating vehicle in the outer circulatory lane is circulating for a vehicle entering the roundabout from the right approach lane.

Time headway: the time between two consecutive circulating vehicles crossing the conflict line. The time headway is *accepted* if a vehicle stopped on the approach enters the roundabout between the two vehicles. The time headway is *rejected* if a vehicle stopped on the approach does not enter the roundabout between the two vehicles.

TABLE 4.4
Sample Size by Studied Factors.

| Condition | Rejected/Accepted Headway | Follow-up Headway |
|--------------------|---------------------------|-------------------|
| Rural area | 544 | 165 |
| Heavy vehicle | 108 | 12 |
| Nighttime/twilight | 121* | 10 |
| Right-lane | 254 | 15 |
| Right-turn | 50 | — |

*Observations are from one rural roundabout.

TABLE 4.6
Data Inventory Format.

| RAB | Approach | Weather | Light | Driver | Headway | Event | NRH | Veh Type | Lane | Turn | Area Type |
|-----|----------|---------|-------|--------|---------|-------|-----|----------|------|------|-----------|
| 4 | 2 | 1 | 1 | 1 | 2.52 | 2 | 0 | 4 | 1 | 1 | 1 |
| 4 | 2 | 1 | 1 | 1 | 2.97 | 2 | 1 | 4 | 1 | 1 | 1 |
| 4 | 2 | 1 | 1 | 1 | 7.68 | 1 | 2 | 4 | 1 | 1 | 1 |
| 4 | 2 | 1 | 1 | 2 | 1.2 | 2 | 0 | 4 | 2 | 2 | 1 |
| 4 | 2 | 1 | 1 | 2 | 6.66 | 1 | 1 | 4 | 2 | 2 | 1 |
| 4 | 2 | 1 | 1 | 3 | 2.2 | 2 | 0 | 4 | 1 | 1 | 1 |
| 4 | 2 | 1 | 1 | 3 | 1.87 | 2 | 1 | 4 | 1 | 1 | 1 |
| 4 | 2 | 1 | 1 | 3 | 1.66 | 2 | 2 | 4 | 1 | 1 | 1 |
| 4 | 2 | 1 | 1 | 3 | 1.48 | 2 | 0 | 4 | 1 | 1 | 1 |
| 4 | 2 | 1 | 1 | 3 | 8.34 | 1 | 4 | 4 | 1 | 1 | 1 |
| 4 | 2 | 1 | 1 | 4 | 2.14 | 2 | 0 | 2 | 1 | 1 | 1 |
| 4 | 2 | 1 | 1 | 4 | 2.28 | 2 | 1 | 2 | 1 | 1 | 1 |
| 4 | 2 | 1 | 1 | 4 | 1.97 | 2 | 2 | 2 | 1 | 1 | 1 |

Follow-up time: the time between two consecutive entering vehicles crossing the yield line (either from a stationary or moving queue) and accepting the same time headway between circulating vehicles.

Based on the recorded time stamps at the specific conditions described above, the headways were calculated as follows:

$$\text{Rejected headway} = \text{Time2} - \text{Time1}$$

$$\text{Accepted headway} = \text{Time5} - \text{Time2}$$

$$\text{Follow-up headway} = \text{Time4} - \text{Time3}$$

TABLE 4.5
Variables Available to Estimate Critical Headways.

| Variable No. | Variable Description |
|--------------|--|
| 1 | Measured Headway (sec) |
| 2 | Event (decision): 1 if accepted, 2 if rejected, 3 if follow-up |
| 3 | Number of Rejected Headways (as proxy to congestion level) |
| 4 | Vehicle Type: 1 if car or pickup, 2 if Single Unit Truck, 3 if Bus, 4 if Trailer, 5 if other types (e.g., motorbike) |
| 5 | Approach Speed: 1 if high-speed, 2 if low-speed |
| 6 | Lane Use: 1 if left, 2 if right |
| 7 | Turning Maneuver: 1 if through/left/U-turn, 2 if right |
| 8 | Lighting Condition: 1 if daytime, 2 if twilight, 3 if nighttime |
| 9 | Weather Condition: 1 if no rain, 2 if rainy |
| 10 | Area Type: 1 if urban, 2 if rural |

The data set extracted from the video footage contains 2,899 observations for critical headway and 813 observations for follow-up headway estimations. The observations are broken down by roundabout in Table 4.3 and by studied factors in Table 4.4. The available variables for model estimation are shown in Table 4.5.

Finally, the extracted data were organized in a usable format for future research work. Table 4.6 shows a sample of the data inventory format. The codes used for the explanatory variables are as described in Table 4.5.

5. ROLLOVER PROPENSITY ANALYSIS

The method of observing drivers' behavior and estimating the rollover propensity was discussed in the previous chapters. This chapter applies this research method to investigate the trucks' rollover propensity at selected Indiana roundabouts to answer several relevant questions. The first and most critical question is if indeed drivers of heavy vehicles experience an increased risk of rollover when negotiating the roundabouts built on Indiana high-speed roads. The first section of this chapter deals with this question by comparing the behavior of truck drivers and their propensity for rollover on selected roundabouts located on high and low-speed roads.

Selected facets of rollover at roundabouts are also discussed in the attempt to identify conditions when the rollover propensity might be heightened. Such discovery would reveal opportunities for proactive safety improvement. These additional aspects include: circulatory roadway superelevation, drivers' aggressive behavior, roundabout readability, and nighttime conditions.

As discussed in the previous section, the weight of the studied semi-trailers was unknown. In the first section examining roundabouts on low and high-speed roads, both the unloaded and loaded trailer assumptions are considered. The remaining sections consider only the loaded trailer assumption; in addition to being the most critical case (smallest Δv), the majority of carriers maintain full or nearly full trailers to reduce shipping costs.

5.1 Low and High-Speed Roads

One of the key concerns about roundabouts on high-speed roadways is the ability of drivers to slow their vehicles in order to safely maneuver the roundabout. Figure 5.1 and Figure 5.2 display the actual speeds of a randomly selected sample of drivers in relation to the yield line for roundabouts on low and high-speed roads, respectively. These roundabouts are the Concord Road/Maple Point Drive (low-speed) and State Road 25 (high-speed) roundabouts in Lafayette. Speeds are presented at distances up to 400 feet upstream of the

roundabout yield line. Drivers on the low-speed approach began their deceleration at around 350 feet upstream of the roundabout yield line, whereas the drivers on the high-speed approach began their deceleration well before this point around 800–900 feet upstream of the yield line. Within 350 feet of the yield line, drivers on both approaches maintained similar deceleration rates, decreasing their speed by around 6 mph every 100 feet until reaching similar speeds on the roundabout approach curve. The results indicate that drivers on the high-speed approach began their deceleration earlier than drivers on the low-speed approach. Eventually, the high-speed and low speed approaches exhibit consistent driver speed profiles at the distance closer than 350 ft from the roundabout yield line.

After traversing the approach curve, drivers at both roundabouts tended to slow gradually until most reached their minimum speed on the circulatory roadway. However, Figure 5.1 shows that some drivers on the low-speed roadway slowed down on the approach to a speed so low that they had to accelerate before reaching the yield line. Since the drivers were unaffected by external influences, this behavior might be due to the driver misperceiving the roundabout geometry or miscalculating a conflict with a vehicle in the circulating lane. As seen in Figure 5.2, there was also a more rapid dip to minimum speed for circulating drivers on the high-speed roadway, which again may be attributed to the geometrical characteristics of the roundabout.

Figure 5.3 and Figure 5.4 present the paths of the trailer center of mass for a randomly selected subset of the observed drivers. The square symbols on these paths mark the locations where Δv_a and Δv_c values reached their minimum values on the roundabout approach and the circulatory roadway, respectively. At both the roundabouts, the lowest Δv_c primarily occurred near the end of the circulation when the driver in the cab began to straighten out and accelerate but the trailer center of mass was still on the curve. Contrasting the two roundabouts, Figure 5.3 shows less scatter in the locations of minimum Δv_c for the roundabout on the low-speed road. This pattern could be due to its single-lane geometry, which constrained driver path selection.

Table 5.1 summarizes the results. The lowest Δv_a and corresponding actual speeds v_a were only slightly lower on the low-speed road. Although the radius of the approach curve on the low-speed roundabout was smaller, the radii of the paths selected by drivers on low-speed and high-speed approaches were similar as seen in Figure 5.5. It can be concluded that the speed difference observed 800–900 feet upstream of the two studies roundabouts had a minimal if any impact on the behavior of drivers when entering the two roundabouts.

Given the lack of behavioral difference on the two approaches close to the roundabouts, it may be surprising that a difference was found in the roundabout circulation. The roundabout on the low-speed road had the average Δv_c smaller than the roundabout

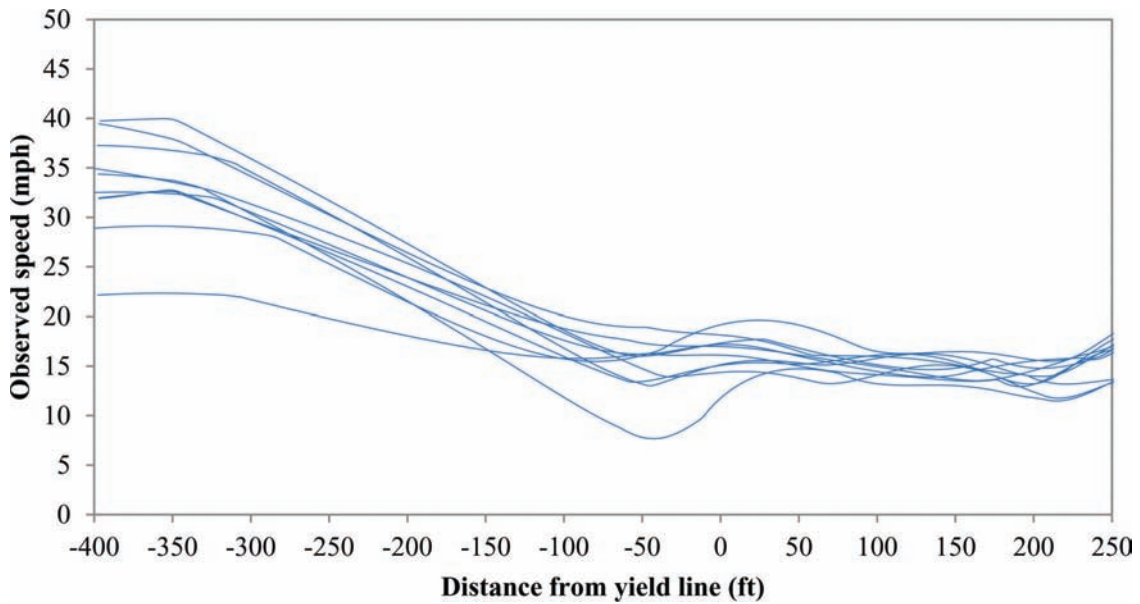


Figure 5.1 Actual speed and distance from roundabout yield line for sample of drivers on low-speed road.

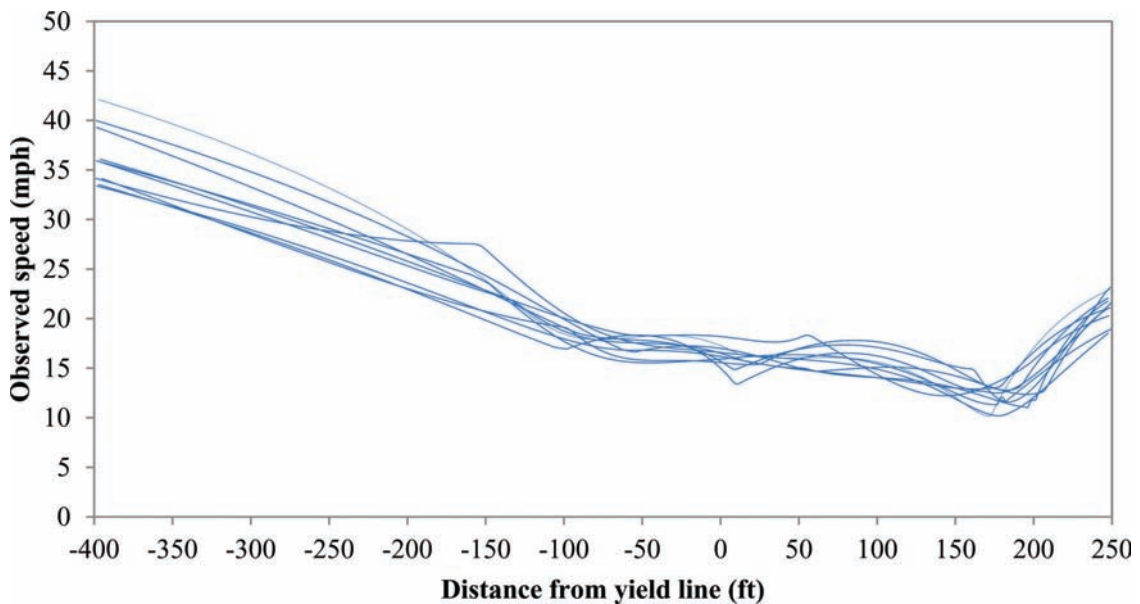


Figure 5.2 Actual speed and distance from roundabout yield line for sample of drivers on high-speed road.

on the high-speed road. This difference is considerable and statistically significant and it indicates a closer proximity to rollover on the low-speed road.

To explain the difference in the turnover propensity on the studied circulation roadways, radii selected by drivers are compared between the two roundabouts in Figure 5.6. These radii are considerably different. The roundabout on the low-speed road, which has a single circulatory lane, apparently constrained the path radius selection leading to a much narrower range of radii implemented by the drivers. At the same time, the two-

lane roundabout on the high-speed road allowed drivers much greater flexibility in path selection leading to a much stronger variability of the selected radii. Restricting the path radius reduces the driver's ability to compensate for a higher speed with a longer radius. This is a plausible explanation why the marginally lower actual speed v_c may be associated with the closer proximity to rollover at the single-lane roundabout.

The lowest Δv_a and Δv_c values are compared in Figure 5.7 and Figure 5.8 assuming loaded and unloaded trailers, respectively. The minimum Δv_a were

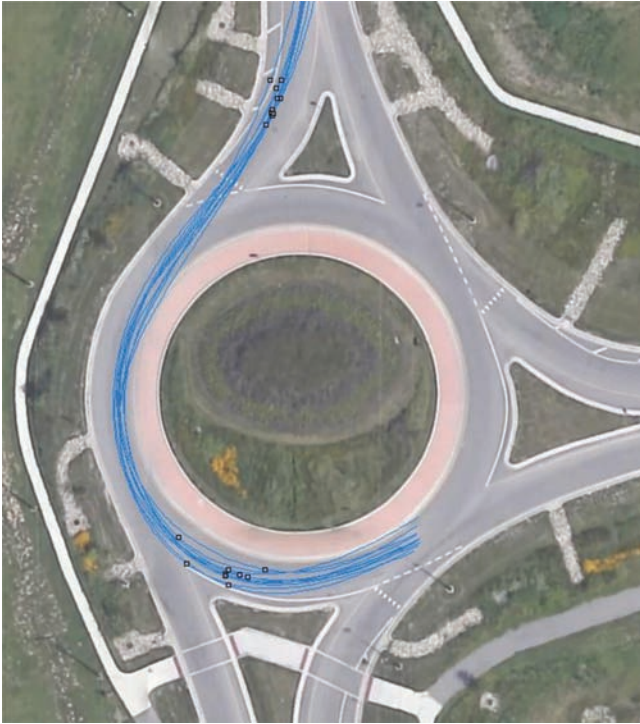


Figure 5.3 Path of center of mass and locations of minimum Δv_a and Δv_c for sample of drivers on low-speed road (Google Maps).



Figure 5.4 Path of center of mass and locations of minimum Δv_a and Δv_c for sample of drivers on high-speed road (Google Maps).

similar for both roundabouts. However, the lower Δv_c at the roundabout on the low-speed roadway shows that the most safety critical maneuvers occurred within its circulatory roadway. This latter difference has been associated with a lower freedom of selecting a path on the single-lane circulatory roadway.

These results lead to two important conclusions:

- The propensity for rollover at a roundabout may not be affected by the speed limit on the approach road if the roundabout is well designed and is readable from a sufficient distance. The entry speeds tend to be similar regardless of the speed limits on the crossing roads if the approach and circulatory curves are similar.
- A wider circulatory roadway offers more freedom of selecting a path by drivers of heavy vehicles. It may help compensate for a higher speed on the circulatory road by selecting a longer radius.

5.2 Circulatory Superelevation

As observed in the previous section, two locations stand out where rollover is of particular concern. These tend to occur where the horizontal radius of the roadway is smallest: the approach curve and circulatory roadway. Figure 5.9 presents the cumulative distribution of the minimum Δv values on both the approach curve and circulation for the sample vehicles at the State Road 25 roundabout. From this, it is clear that the circulatory roadway is the more critical location for heavy vehicle rollover propensity. Hence, the vehicle trajectories along this portion were selected for further analysis.

The current design practices in the United States favor using an outward superelevation (Gingrich & Waddell, 2008), often at a 2% slope. Given the benefits of inward sloping roadways in reducing the rollover

TABLE 5.1
Average Rollover Propensity on Approach Δv_a (mph) and on the Circulatory Roadway Δv_c (mph).

| Approach Type | Δv_a | | Observed Speed, v_a | Δv_c | | Observed Speed, v_c |
|---------------|--------------|----------|-----------------------|--------------|----------|-----------------------|
| | Loaded | Unloaded | | Loaded | Unloaded | |
| Low-speed | 19.58 | 29.91 | 15.95 | 6.55 | 12.19 | 13.60 |
| High-speed | 20.07 | 30.58 | 16.78 | 9.13 | 15.54 | 14.28 |
| Difference | -0.49 | -0.67 | -0.83 | -2.58 | -3.35 | -0.68 |
| t-value | 0.64 | 0.66 | 0.91 | 5.87 | 6.93 | 1.23 |

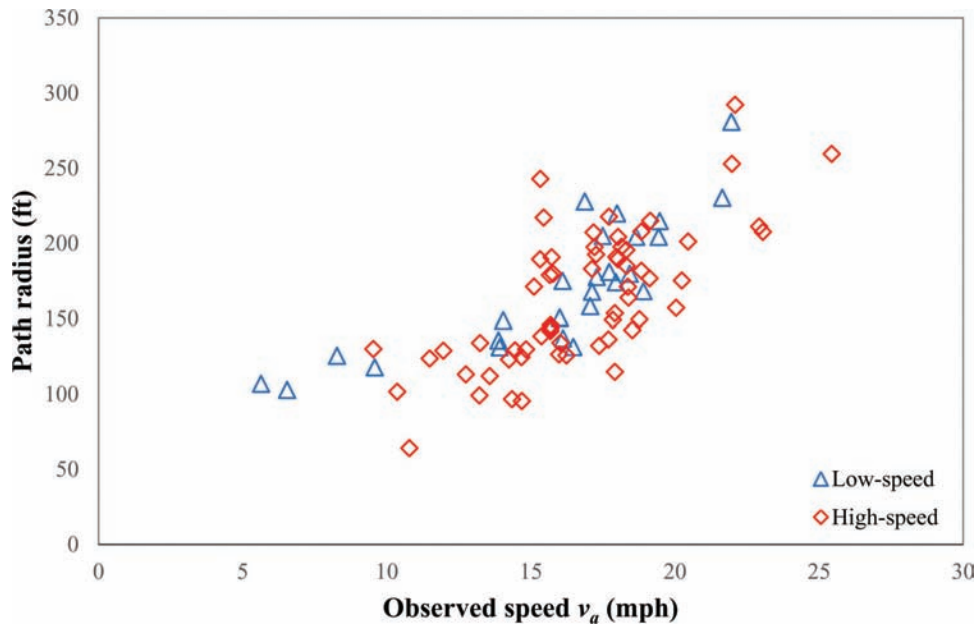


Figure 5.5 Path radius and actual speed v_a for each driver on low and high-speed roads.

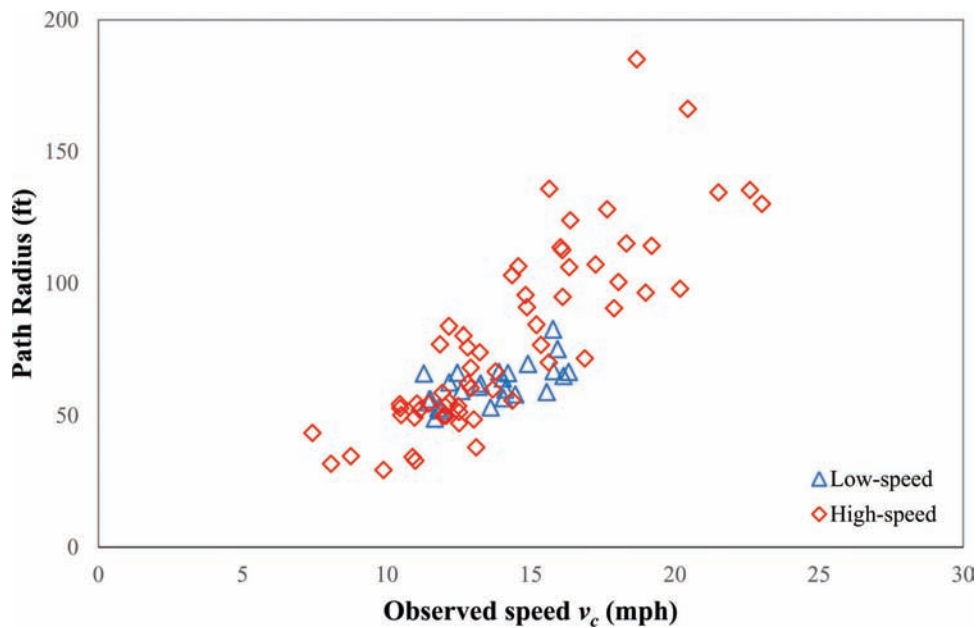


Figure 5.6 Path radius and actual speed v_c for each driver on low and high-speed roads.

risk, the commonly used 2% outward slope and alternative of 2% inward slope were assumed for the studied roundabout to quantify the proximity to rollover between these two alternatives. An assumption was made that limited changes in the pavement elevation are not noticeable by truck drivers, or do not affect truck driver behavior as would be evidenced in their selection of path and speed (Gingrich & Waddell, 2008). This assumption allowed estimating

the threshold speeds and corresponding Δv values for the two pavement elevation scenarios using the observed trajectories for 63 vehicles. Figure 5.10 displays graphically the distribution of minimum Δv_c values during circulation for the two studied scenarios and for the third scenario with 3% inward slope.

As the inward superelevation increases, the tendency for a cornering vehicle to rollover is expected to decrease. It has been confirmed by analyzing the case

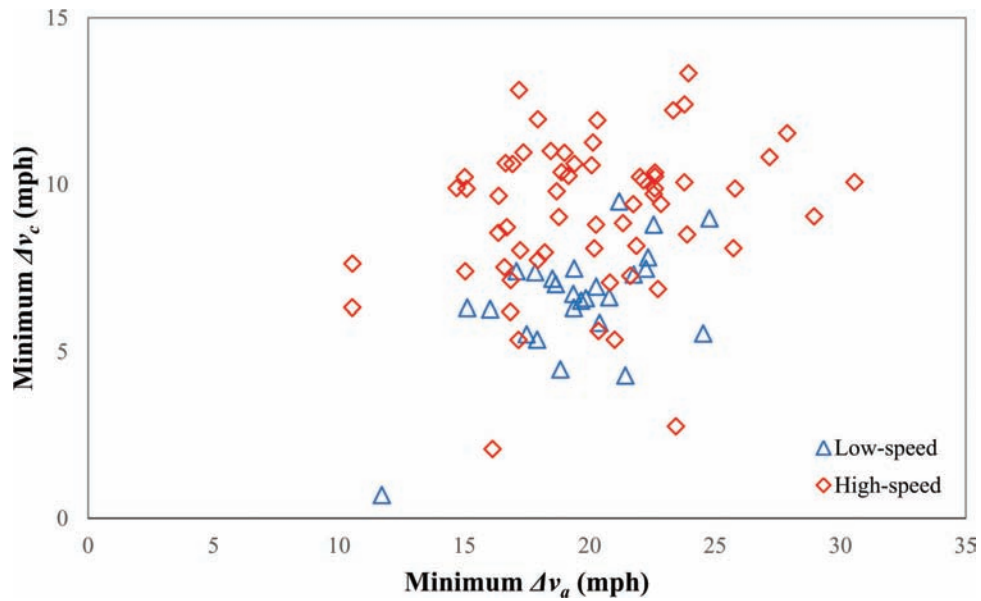


Figure 5.7 Minimum Δv_c and Δv_a for each driver on low and high-speed roads, loaded trailers.

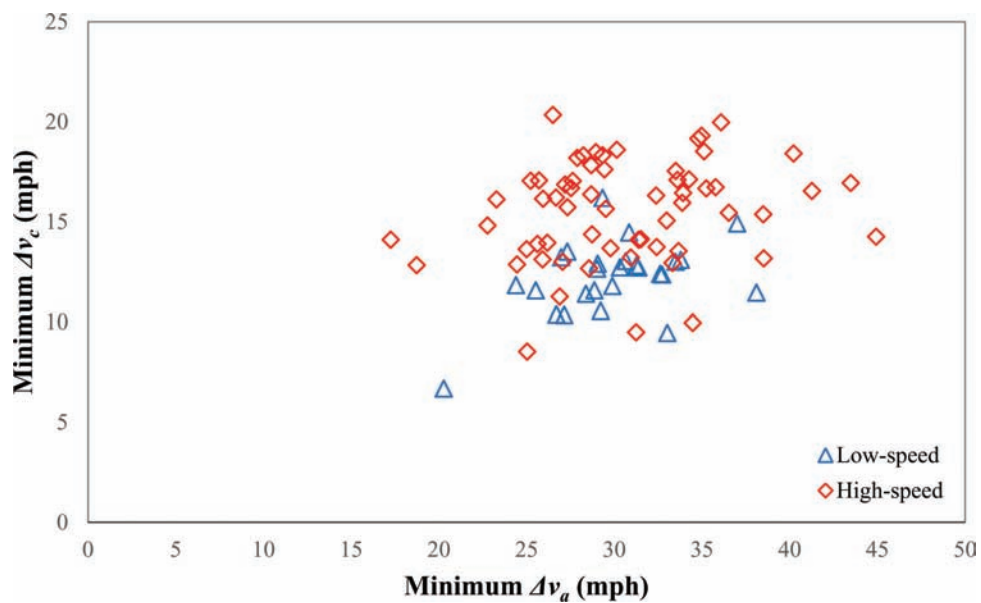


Figure 5.8 Minimum Δv_c and Δv_a for each driver on low and high-speed roads, unloaded trailers.

with 3% inward superelevation. Table 5.2 provides a summary of the results.

The difference between the minimum Δv_c values for outward and inward superelevations was tested with the t statistic applied to paired observations (two pavement elevation scenarios for each vehicle). Both comparisons produced highly significant t statistics. The comparison of the 2% inward and 2% outward superelevation scenarios produced the t value of 57.42, while the comparison of the 3% inward and 2% outward scenarios yielded a value of 59.69.

The results suggest that inward circulatory superelevation indeed decreases rollover propensity. However, this decrease is small and may not justify changing the current design practice because the inward sloping superelevation introduces other considerable challenges. Relatively abrupt changes in cross fall between the approach curve and circulation, in addition to drainage issues, are challenges of the design. The former has been confirmed by previous studies as increasing rollover propensity (Highways England, 2007). The latter results from having to drain water

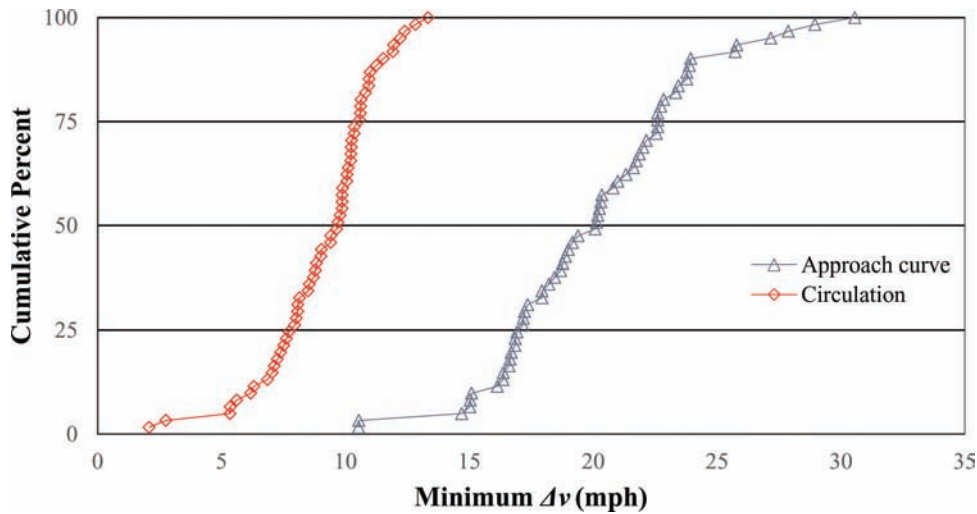


Figure 5.9 CDF of minimum Δv for approach and circulation curves.

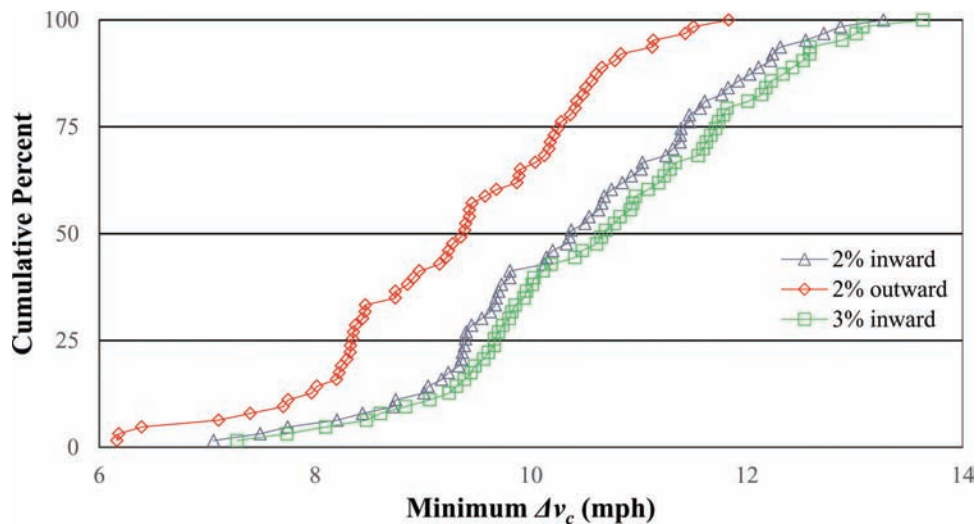


Figure 5.10 CDF of minimum Δv_c for superelevation scenarios.

TABLE 5.2
Comparison of Mean Minimum Δv_c (mph) for Superelevation Scenarios.

| Superelevation | | | Difference | |
|---------------------------|-----------|------------|--------------------------|--------------------------|
| 3% Inward | 2% Inward | 2% Outward | 2% Inward vs. 2% Outward | 3% Inward vs. 2% Outward |
| Mean Minimum Δv_c | | | | |
| 10.68 | 10.41 | 9.29 | 1.12 | 1.39 |

from the roundabout's center, which could especially be an issue during the cold season when freezing may cause icy road conditions. Coupled with indications of higher crash rates resulting from an inward slope (Jacquemart, 1998), we conclude that there is no strong

basis to discontinue the common practice of using outward circulatory superelevation.

5.3 Aggressive Behavior

Certain drivers are prone to aggressive behavior, such as driving at excessive speeds. To determine if this behavior correlates with a decreased rollover margin at the roundabout, drivers should be classified according to their actual speed far from the roundabout's influence. As such, the farthest distance at which speed can be reasonably estimated based on the conditions of this study was selected, which is 800 ft from the roundabout yield line. Vehicles at the State Road 25 roundabout in Lafayette were studied. Their speeds measured at 800 ft from the yield line were grouped into

the speeds above the 75th-percentile (aggressive behavior) and below the 25th-percentile (passive behavior). The behavior of these two groups at the roundabout was compared.

As the circulation has previously been confirmed as the most critical location for rollover, Δv_c was computed here for each of the studied vehicles. Figure 5.11 shows the comparison between minimum Δv_c for the above 75th- and below 25th-percentile speed groups.

A *t* test is performed to determine the statistical significance of the difference between mean minimum Δv_c . The results are displayed in Table 5.3.

The results of the *t* test indicate drivers displaying more aggressive behavior, based on speed, come marginally closer to the critical rollover threshold at the roundabout circulation. However, the *t* statistic does not indicate significance at typical confidence levels. Based on the conditions of this study, there is no clear pattern between aggressive driver behavior and a tendency for encroaching on the rollover threshold at the roundabout.

5.4 Roundabout Readability

In close proximity to a roundabout, a sudden change of the deceleration rate may be interpreted as a corrective action after misreading the roundabout geometry at the farther distance. In order to detect such a behavior, two deceleration rates were measured: a_i implemented by a driver in advance of the roundabout shortly after the braking was initiated and a_f just before the roundabout approach curve. The difference between these rates was calculated as $\Delta a = a_f - a_i$.

It is postulated that a driver deceleration immediately upstream of the approach curve that exceeds a

comfortable rate could indicate an error in perceiving the roundabout geometry and lead to an increased risk of rollover. A deceleration rate for trucks on dry pavement is considered comfortable if it is less in magnitude than 5 ft/s² (Harwood et al., 2003). The percentage of drivers who applied deceleration rates in excess of the 5 ft/s² threshold and the average Δv_a on the approaches was estimated for drivers at the State Road 25 and Concord Road/Maple Point Drive roundabouts in Lafayette. Several different conditions were considered including the speed on the approach road (low and high-speed), type of maneuver at the roundabout (through and left-turn movements), and the time of day (daytime and nighttime). The results are displayed in Table 5.4, Table 5.5, and Table 5.6, respectively.

The results in Table 5.4 indicate that truck drivers at the roundabout on high-speed road tended to use a greater deceleration rate a_f near the roundabout approach curve than drivers on the low-speed road. As the result of these different behaviors, the actual speeds v_a and the Δv_a values were similar in the two examined scenarios.

Table 5.5 shows that the deceleration rate a_f used by drivers making through or left-turn movements were

TABLE 5.3
Mean Minimum Δv_c (mph) at Roundabout Circulation for Drivers with Upper 75th- and Lower 25th-Percentile Speeds at 800 ft from the Yield Line.

| Approach Speed Classification | Δv_c |
|-----------------------------------|--------------|
| Below 25 th percentile | 9.17 |
| Above 75 th percentile | 8.98 |
| t-value | 0.22 |

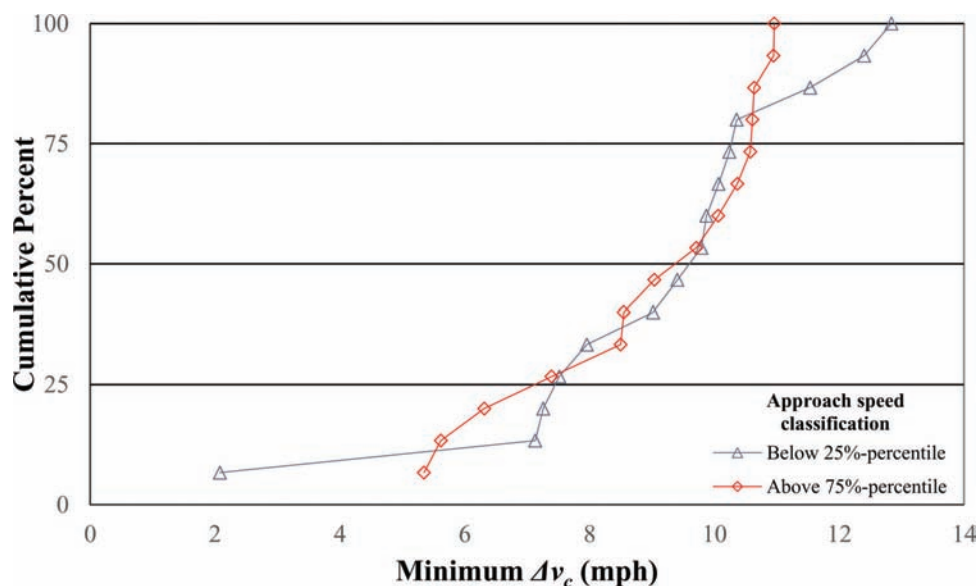


Figure 5.11 Minimum Δv_c at roundabout circulation for drivers with upper 75th- and lower 25th-percentile speeds at 800 ft.

similar. The Δv_a and corresponding v_a were slightly larger for the through movements, suggesting drivers selected a larger path radius on the approach curve compared to left-turn movements in order to increase the rollover margin.

Table 5.6 shows that drivers had only a slightly smaller deceleration rate a_f at nighttime as compared to daytime. This is consistent with the larger Δv_a at night, which suggests more conservative driving behavior than during the day.

Change in the deceleration rate Δa during braking on an approach road is another indication that a driver

might be correcting his/her maneuver when getting closer to the approach curve. The magnitude of the correction may be associated with the propensity for rollover on the approach curve. Figure 5.12 displays Δa versus the corresponding minimum Δv_a on the roundabout approach curve for drivers at the State Road 25 roundabout in Lafayette. A driver increasing his braking rate near the roundabout is indicated by a negative value of Δa . The connection between Δa and Δv_a was non-existent or weak at best.

Figure 5.13 shows the deceleration rate a_f and the corresponding minimum Δv_a on the roundabout

TABLE 5.4
Mean Deceleration Rate a_f (ft/s²) and Percentage Below Comfort Threshold for Roundabouts on Low- and High-Speed Roads.

| Approach Type | Average Deceleration Rate, a_f | Percentage below Threshold | Δv_a | Actual Speed, v_a |
|-----------------|----------------------------------|----------------------------|--------------|---------------------|
| Low-speed | -2.63 | 7.69 | 19.58 | 15.95 |
| High-speed | -3.57 | 21.67 | 20.07 | 16.78 |
| Difference | 0.94 | -13.98 | -0.49 | -0.83 |
| <i>t</i> -value | 2.37 | | -0.64 | -0.91 |

TABLE 5.5
Mean Deceleration Rate a_f (ft/s²) and Percentage Below Comfort Threshold for Roundabouts with Through and Left-Turn Movements.

| Movement Type | Average Deceleration Rate, a_f | Percentage below Threshold | Δv_a | Actual Speed, v_a |
|-----------------|----------------------------------|----------------------------|--------------|---------------------|
| Through | -2.74 | 9.68 | 21.17 | 16.98 |
| Left | -2.63 | 7.69 | 19.58 | 15.95 |
| Difference | -0.11 | 1.99 | 1.59 | 1.03 |
| <i>t</i> -value | -0.28 | | 1.85 | 0.98 |

TABLE 5.6
Mean Deceleration Rate a_f (ft/s²) and Percentage Below Comfort Threshold under Nighttime and Daytime Conditions.

| Condition | Average Deceleration Rate, a_f | Percentage below Threshold | Δv_a | Actual Speed, v_a |
|-----------------|----------------------------------|----------------------------|--------------|---------------------|
| Nighttime | -3.20 | 18.18 | 23.36 | 16.52 |
| Daytime | -3.57 | 21.67 | 20.07 | 16.78 |
| Difference | 0.37 | -3.49 | 3.29 | -0.26 |
| <i>t</i> -value | 0.96 | | 4.36 | -0.41 |

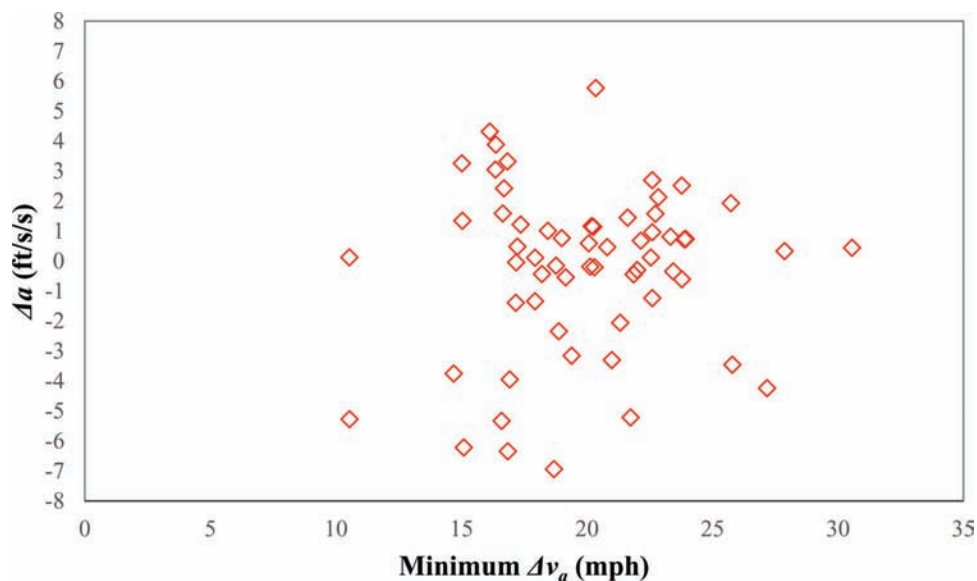


Figure 5.12 Δa and minimum Δv_a for each studied driver at State Road 25 roundabout.

approach curve for drivers at the State Road 25 roundabout. Drivers with greater (more negative) deceleration rates were generally observed to come closer to the critical rollover threshold on the roundabout approach curve.

Furthermore, drivers were separated based on the percentile of their deceleration rates in close proximity to the roundabout approach curve. Lower 25th-percentile versus upper 75th-percentile deceleration rates were the examined scenarios. Δv_a was then compared at the approach curve for the studied vehicles (see Figure 5.14).

Table 5.7 confirms a considerable difference in mean minimum Δv_a among the examined scenarios.

5.5 Nighttime Conditions

The effects of reduced visibility and possible fatigue of truck drivers in nighttime conditions is an important consideration, particularly for Northern Latitude cities where these conditions are frequent during rush hours in the late fall and winter months.

The application of the previously derived model is demonstrated using daytime and nighttime data collected from the State Road 25 roundabout in Lafayette. The comparison of the Δv_a and Δv_c values for each vehicle is shown in Figure 5.15. There is no obvious consistency of driver behavior on the approach and on the circulatory roadway. In other words, aggressive drivers with low Δv_a on the approach do not tend to have low Δv_c on the circulatory roadway. The same lack of consistency is also seen for drivers with large Δv_a on the approach. This disconnection between the behavior on the approach and on the circulatory road occurs in both the daytime and nighttime conditions.

The main pattern that may be noticed in Figure 5.15 is the generally more cautious behavior of drivers on the approach in the nighttime conditions demonstrated through a slight shift of the cloud of nighttime points along the horizontal axis towards the larger Δv_a values. The difference between the average values of Δv_a in the nighttime and daytime conditions is presented in Table 5.8. The t value of the difference under this assumption is 4.36, indicating that the difference in the behavior between day and night is statistically significant. It is also rather negligible from the safety point of view given the limited differences and the large margins of safety. Consideration for the circulatory roadway showed a similar pattern, but not statistically significant.

Comparison of the Δv_a and Δv_c distributions across vehicles in the daytime and nighttime conditions is presented in Figure 5.16 for the roundabout approach and in Figure 5.17 for the circulatory roadway. Figure 5.16 clearly indicates that the truck drivers as a group were more cautious when approaching the roundabout at night than during the day. This pattern is also observed in Figure 5.18, where for similar actual speeds, there is a small upward shift in the radius of the vehicle path under nighttime conditions. Figure 5.19 presents a less clear trend. The behavioral difference on the circulatory roadway was smaller than on the approach and not statistically significant. As previously noted, the roundabout had street lighting installed on the approach and circulation. Despite this, indications are that drivers act more conservatively at night while perceiving and anticipating the roundabout and adjust to a certain degree after entering the circulation.

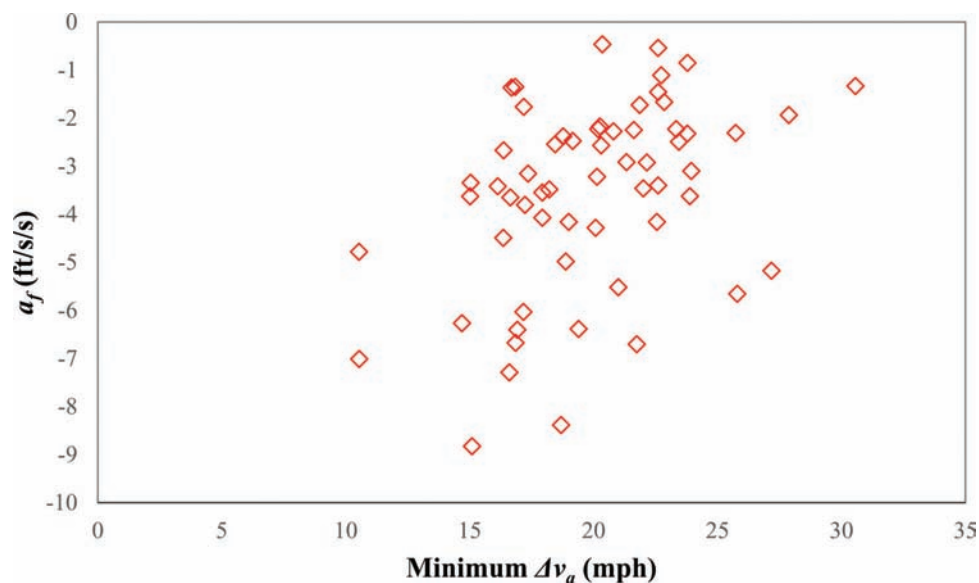


Figure 5.13 a_f and minimum Δv_a for each studied driver at State Road 25 roundabout.

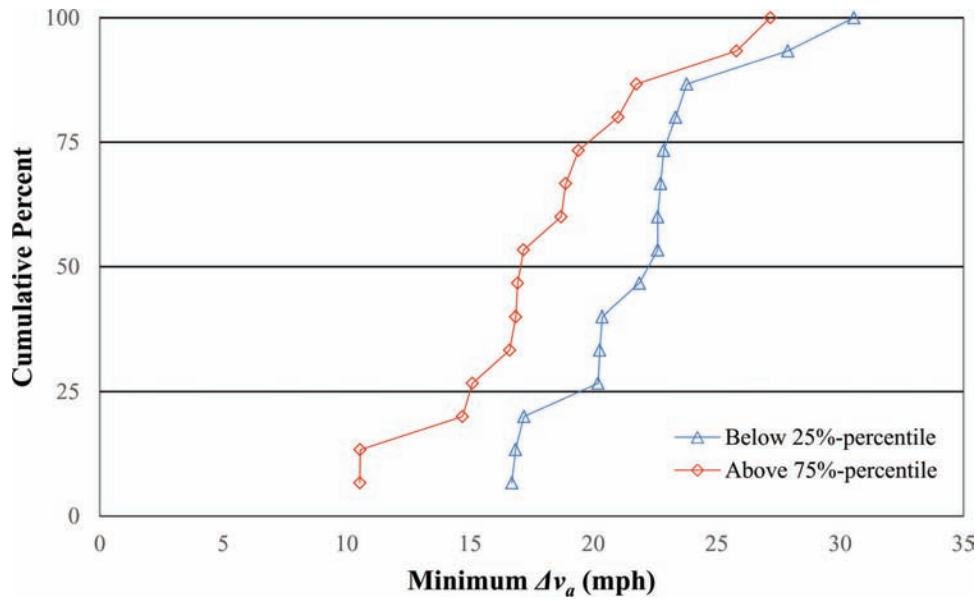


Figure 5.14 Minimum Δv_a for drivers with upper 75th- and lower 25th-percentile deceleration rates a_f at State Road 25 roundabout.

TABLE 5.7
Mean Minimum Δv_a for Drivers with Upper 75th- and Lower 25th-Percentile Deceleration Rates a_f at State Road 25 Roundabout.

| Deceleration Rate Classification | Δv_a |
|-----------------------------------|--------------|
| Below 25 th percentile | 21.99 |
| Above 75 th percentile | 18.08 |
| t-value | 2.51 |

TABLE 5.8
Average Rollover Propensity on Approach Δv_a (mph) and on the Circulatory Roadway Δv_c (mph).

| Condition | Δv_a | Actual Speed, v_a | Δv_c | Actual Speed, v_c |
|------------|--------------|---------------------|--------------|---------------------|
| Nighttime | 23.36 | 16.52 | 9.92 | 13.72 |
| Daytime | 20.07 | 16.78 | 9.13 | 14.29 |
| Difference | 3.29 | | 0.79 | |

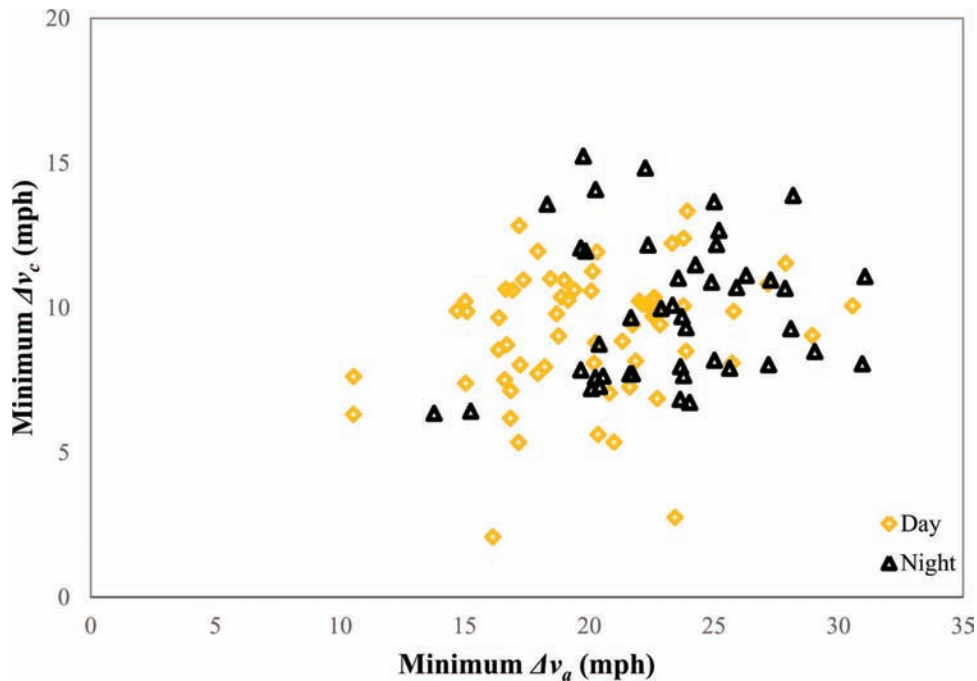


Figure 5.15 Minimum Δv_a on circulatory roadway and approach curve for each driver during daytime and nighttime conditions.

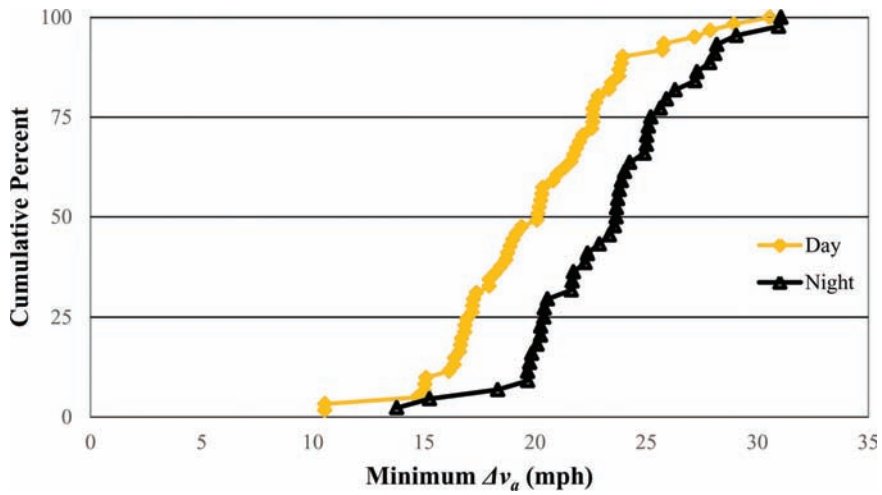


Figure 5.16 CDF of minimum Δv_a on approach curve during daytime and nighttime conditions.

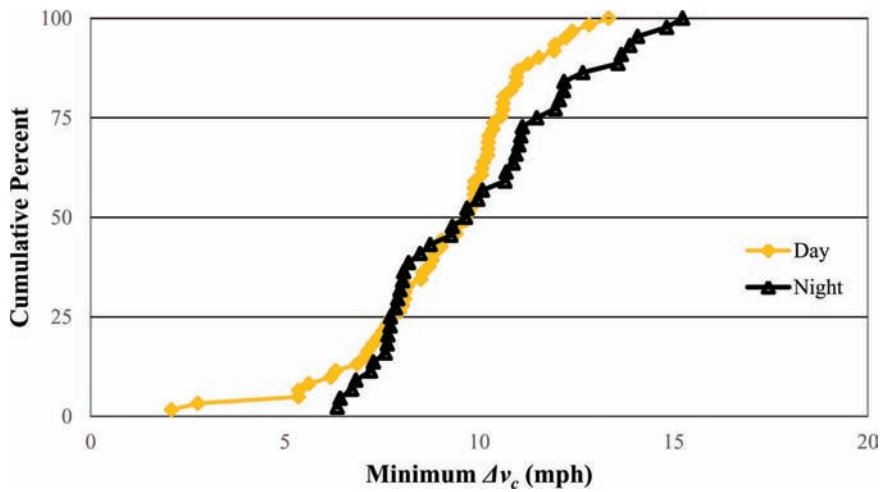


Figure 5.17 CDF of minimum Δv_c on circulatory roadway during daytime and nighttime conditions.

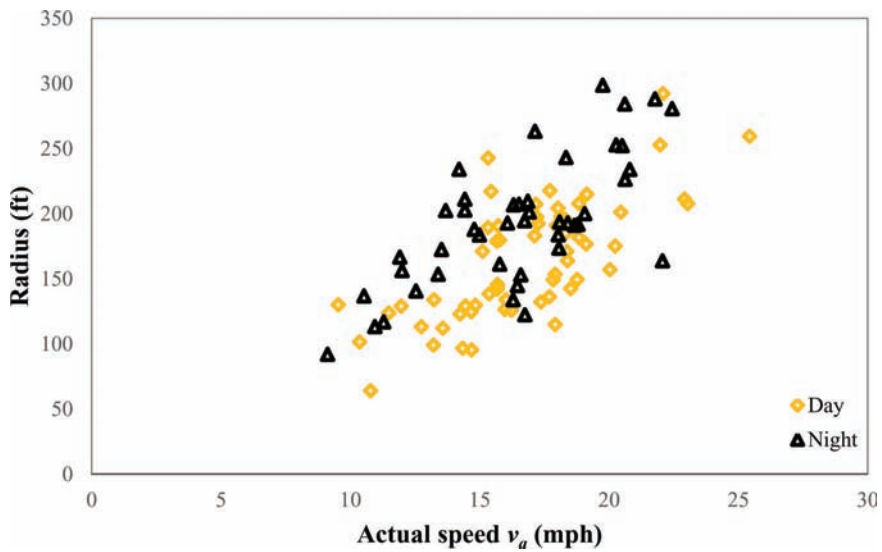


Figure 5.18 Radius and actual speed v_a for each driver during daytime and nighttime conditions.

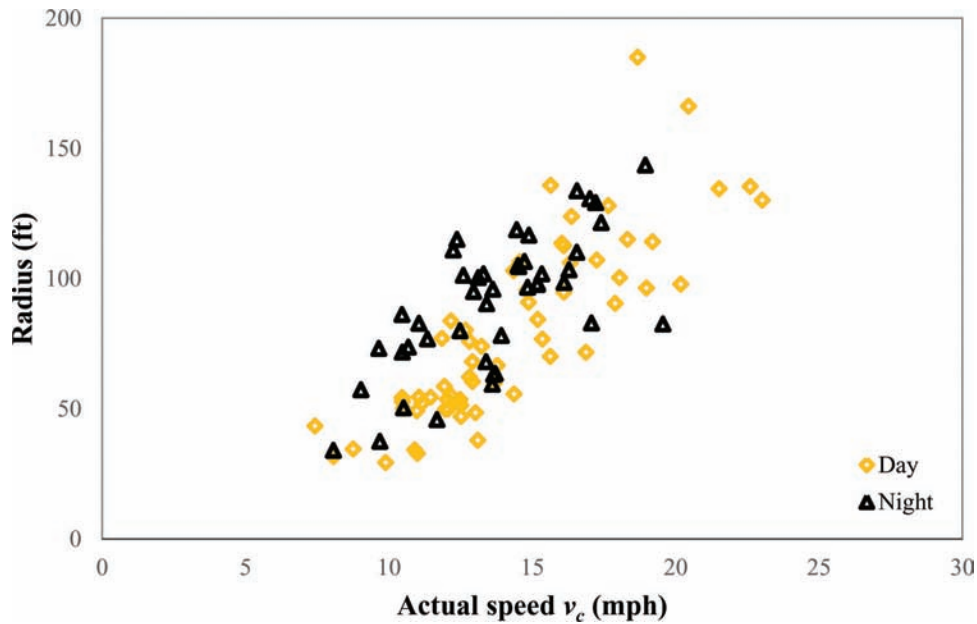


Figure 5.19 Radius and actual speed v_c for each driver during daytime and nighttime conditions.

6. CAPACITY ANALYSIS

6.1 Binary Probit Model for Critical Headways

SAS statistical software was used to estimate the model for critical headways. The estimated binary Probit model is shown in Table 6.1. The base conditions were defined as a passenger car, low-speed approach in an urban area, daylight, and single-lane roundabout. The independent variables that were found to be statistically significant at a 5% significance level were measured headway, dual-lane in rural area, heavy vehicles, nighttime/twilight conditions, and number of rejected headways (as a proxy variable for congestion level). The constant (intercept) was also significant.

The effects of the significant variables on critical headway are quantified using Equations 3.13 and 3.14 and summarized in Table 6.2.

Although the estimated model revealed the fact that the driver behavior is affected by the number of rejected headways, it is more convenient to have one model to normalize this effect, for practice purposes. Therefore, the NRH indicator variable is excluded from the model. For the estimated model without this variable refer to Appendix C (Table C.1). Consequently, the base-case critical headway was estimated 4.4 sec (as opposed to 4.7 sec). The cumulative distribution function of the estimated critical headways (normal distribution with mean, μ , 4.4, and standard deviation, σ , 1.0) for the

TABLE 6.1
Binary Probit Model for Critical Headway Estimation.

| Variable | Parameter Estimate | t-value |
|--|--------------------|---------|
| Constant (intercept) | -4.775 | -27.44 |
| Measured headway | 1.016 | 26.00 |
| Dual-lane in rural area | -0.545 | -3.77 |
| Heavy vehicles (trucks and buses) | -1.015 | -3.61 |
| Nighttime/twilight (in the presence of street lighting) | -1.202 | -2.84 |
| Number of rejected headways (as proxy to congestion level) | 0.511 | 5.77 |
| Number of Observations | 2,894 | |
| Maximum Likelihood at Convergence | - 512.360 | |
| McFadden adjusted ρ^2 | 0.696 | |

TABLE 6.2
Effects of the Influencing Factors on Critical Headways.

| Variable | Sample Size | Effect | Magnitude (sec) |
|--|-------------|------------|-----------------|
| The base-case condition (single-lane, urban area, passenger car, daylight) | 1153 | Base | 4.7 |
| Dual-lane in rural area | 544 | Increasing | 0.5 |
| Heavy vehicles (trucks and buses) | 108 | Increasing | 1.0 |
| Nighttime/twilight (in the presence of street lighting) | 121 | Increasing | 0.7* |
| Number of rejected headways (as proxy to congestion level) | 968 | Decreasing | 0.5 |

*The clear difference between nighttime and daylight is 1.2 (rural daytime) – 0.5 (rural nighttime) = 0.7sec, because the data were collected from a rural roundabout only.

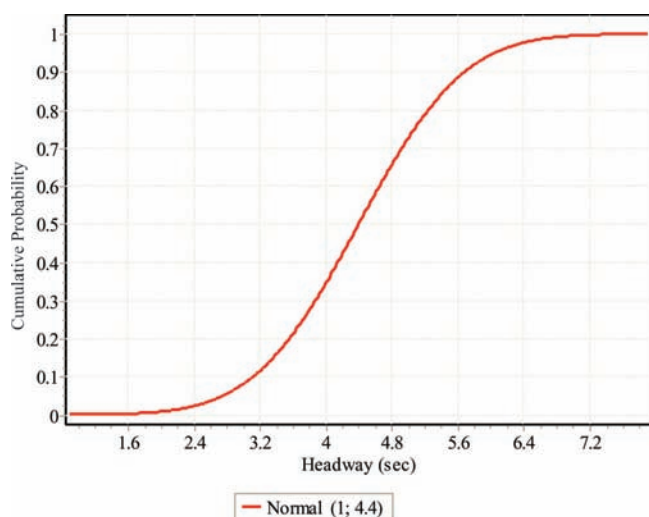


Figure 6.1 Cumulative distribution function of the estimated critical headways for the base-case condition based on Probit model.

base-case condition is shown in Figure 6.1. The estimated critical headways for other conditions along with the MLM results and NCHRP Report 572 findings are summarized in Table 6.3.

6.2 MLM Results for Critical Headways

The estimation for critical headways was repeated using the MLM procedure recommended by Troutbeck (2014), for comparison purposes. The original sample was divided into separate scenarios to estimate the means and standard deviations of the critical headways for the base case, dual-lane in rural area, heavy vehicles, and nighttime/twilight conditions. As the MLM requires pairwise observations, only the largest rejected

TABLE 6.3
Estimated Critical Headways—MLM, Probit Method, and NCHRP 572 Findings.

| Condition | Critical Headway (sec) | | |
|---|------------------------|-----------|----------------------|
| | MLM | Probit | NCHRP 572 (HCM 2010) |
| Single-lane | 4.2 (0.8)* | 4.4 (1.0) | 5.1 |
| Dual-lane (right and left) | 4.9 (1.2) | 5.0 (1.0) | 4.2 R and 4.5 L |
| Heavy vehicles (trucks and buses) | 5.3 (0.8) | 5.5 (1.0) | — |
| Nighttime/twilight (in the presence of street lighting) | 5.6 (0.8) | 5.6 (1.0) | — |

*Standard deviations in parentheses.

TABLE 6.4
Summary of Estimated Critical Headways Based on MLM.

| Condition | Sample Size | Critical Headway (sec) |
|---|-------------|------------------------|
| Single-lane in urban area | 1152 | 4.2 (0.8)* |
| Dual-lane in rural area | 316 | 4.9 (1.2) |
| Heavy vehicles (trucks and buses) | 66 | 5.3 (0.8) |
| Nighttime/twilight (in the presence of street lighting) | 60 | 5.6 (0.8) |

*Standard deviations in parentheses.

headway and the accepted headway were considered. Therefore, the congestion effect based on the number of rejected headways could not be estimated. The estimated critical headways based on the MLM are shown in Table 6.4.

The cumulative distribution function of the estimated critical headways (log-normal distribution with mean, μ , 4.2, and standard deviation, σ , 0.8) for the base-case condition is shown in Figure 6.2.

6.3 Follow-Up Headways

The follow-up headways were averaged and are presented in Table 6.5. The average follow-up headways for heavy vehicles and nighttime conditions are based on relatively small sample sizes and require further data in order to make a stronger conclusion. Based on the estimated values, 2.7 sec can be used as a representative follow-up headway for all the studied conditions, but for heavy vehicles.

6.4 Discussion

The results are discussed from three different viewpoints: (1) significance of the influencing factors on driver gap-acceptance behavior, (2) the calibrated HCM 2010 capacity equations for Indiana conditions, and (3) the methodological approach for critical headway estimation.

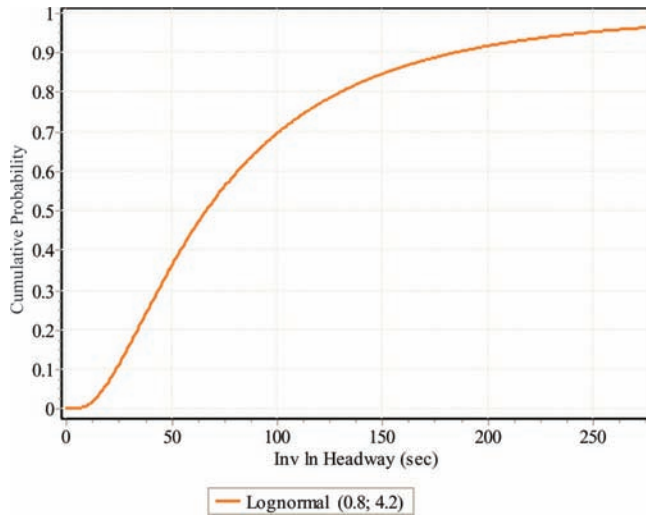


Figure 6.2 Cumulative distribution function of the estimated critical headways for the base-case condition based on MLM.

6.4.1 Capacity Factors

The results indicated that drivers of heavy vehicles (trucks and buses) were likely to accept 1.1 sec longer headways than drivers of passenger cars. Such a result was expected because of truck’s lower acceleration rates and longer lengths require more time to clear the conflict area. Likewise, the difference in the follow-up headways was 0.6 sec. A proper method of accounting for the capacity effects of heavy vehicles is adjusting the service time—the time spent at the first position in queue before entering the roundabout. This method is used in the HCM to calculate the capacity of a traffic lane shared by different turning movements at unsignalized intersections (TRB, 2010). The average service time is calculated from Equation 6.1 separately for passenger cars and heavy vehicles (say trucks) for various circulatory flows, and then the average mixed service time was calculated from Equation 6.2.

Finally, the mixed entry capacity is calculated using Equation 6.3.

$$S_{car} = \frac{1}{C_{car}}, S_{truck} = \frac{1}{C_{truck}} \quad (6.1)$$

$$S_{mix} = P_{car} \cdot S_{car} + P_{truck} \cdot S_{truck} \quad (6.2)$$

$$C_{mix} = \frac{1}{S_{mix}} \quad (6.3)$$

Where: S_{car} , S_{truck} = average service times for cars and for trucks in hours, respectively

C_{car} , C_{truck} = entry capacities for cars and for trucks in veh/h, respectively

P_{car} , P_{truck} = proportions of cars and trucks in the entry lanes, respectively

S_{mix} = average service time for the mixed flow in hours

C_{mix} = entry capacity of the mixed flow in veh/h

The entry capacity values for the mix of 90% passenger cars and 10% heavy vehicles and for various circulatory flows were estimated using the HCM capacity equations with the new estimated gap-acceptance parameters. The obtained capacities for mixed flow are compared to the corresponding capacities of a flow with no trucks in Figure 6.3. The reduced entry capacity for 10% heavy vehicles for various circulatory flows was estimated 7%, on average. This reduction was estimated 12% and 25% for 20% and 50% heavy vehicles, respectively.

As discussed in Chapter 3, the HCM method considers the effects of heavy vehicles by converting the circulating heavy vehicle to a passenger car unit flow rate using an adjustment factor. SIDRA accounts for heavy vehicles by adjusting the critical and follow-up headways. The volume-weighted is another method introduced by Dahl and Lee (2012). The HCM method provides the vehicle adjustment factor of 0.91 calculated for 10% heavy vehicles on a roundabout approach (the same percentage for the circulating traffic) with

TABLE 6.5
Summary of Estimated Follow-Up Headways (sec) for the Studied Conditions.

| Condition | Sample Size | Single Lane | Dual Lane | |
|---|-----------------|-------------|-----------|------------|
| | | Approach | Left Lane | Right Lane |
| Single-lane in urban area | [174, 334, 15]* | 2.7 (0.6)† | 2.7 (0.6) | 2.5 (0.4) |
| Dual-lane in rural area | 20, 41, 135 | 2.6 (0.4) | 2.8 (0.7) | 2.5 (0.8) |
| Heavy vehicles (trucks and buses) | —, 12, —‡ | — | 3.3 (0.9) | — |
| Nighttime/twilight (in the presence of street lighting) | 10, —, — | 2.5 (0.4) | — | — |

*Values correspond to three samples: approach, left-lane, and right-lane; respectively.

†Standard deviations in parentheses.

‡“—” indicates no data.

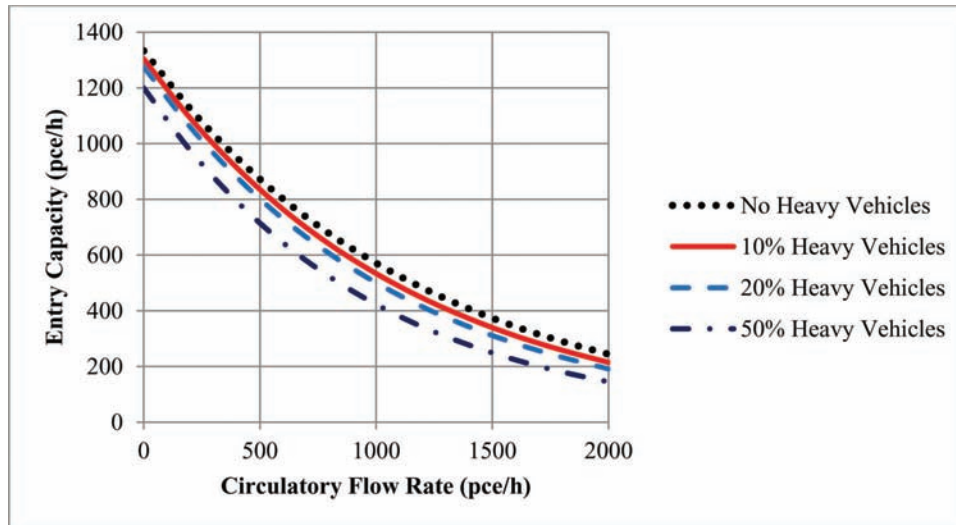


Figure 6.3 Effect of heavy vehicles on the entry capacity for Indiana conditions.

Equations 2.11 and 2.12. The adjusted conflicting flow rates calculated with HCM method are shown in Column 2 of Table 6.6.

On the other hand, the SIDRA method provided the adjusted critical and follow-up headways of 4.4 sec and 2.7 sec, respectively, calculated with Equations 2.13 and 2.14.

$$t'_c = \frac{t_c}{f_{HV}} = \frac{4.4}{0.91} = 4.8\text{sec}, \quad t'_f = \frac{t_f}{f_{HV}} = \frac{2.7}{0.91} = 3.0\text{sec}$$

According to the volume-weighted method, the adjusted critical headways are calculated using Equations 2.15 and 2.16, as below. It was assumed that the follow-up headway for car following car is equal to that of car following truck and similar case for trucks.

$$t'_c = t_{c,C} \cdot (1 - P_{TE}) + t_{c,T} \cdot P_{TE}$$

$$t'_c = 4.4 \cdot (1 - 0.1) + 5.5 \cdot (0.1) = 4.5 \text{ sec}$$

$$t'_f = t_{f,CC}(1 - P_{TE})^2 + (t_{f,CT} + t_{f,TC})(1 - P_{TE})P_{TE} + t_{f,TT} \cdot P_{TE}^2$$

$$t'_f = 2.7(1 - 0.1)^2 + (2.7 + 3.3)(1 - 0.1)0.1 + 3.3 \cdot (0.1)^2 = 2.8\text{sec}$$

Table 6.6 and Figure 6.4 present the entry capacity values, calculated with the three aforementioned methods, for a range of circulating traffic volumes.

As seen in Table 6.6 and Figure 6.4, the HCM method does not consider the fact that heavy vehicles on the approach have larger follow-up headways, thus

TABLE 6.6
Effect of 10% Heavy Vehicles on the Entry Capacity Based on Service Time, HCM, SIDRA, and Volume-Weighted Methods for Indiana Conditions.

| Circulatory Flow | | Entry Capacity (pce/h) | | | | |
|------------------|---------|------------------------|--------------|------|-----------------|-------|
| (veh/h) | (pce/h) | No Heavy Vehicles | Service Time | HCM | Volume-Weighted | SIDRA |
| 0 | 0 | 1330 | 1304 | 1330 | 1280 | 1210 |
| 200 | 220 | 1122 | 1092 | 1103 | 1077 | 1004 |
| 400 | 440 | 947 | 914 | 916 | 907 | 833 |
| 600 | 660 | 799 | 764 | 760 | 763 | 691 |
| 800 | 880 | 675 | 638 | 631 | 642 | 574 |
| 1000 | 1100 | 570 | 534 | 523 | 541 | 476 |
| 1200 | 1320 | 481 | 446 | 434 | 455 | 395 |
| 1400 | 1540 | 406 | 372 | 360 | 383 | 328 |
| 1600 | 1760 | 342 | 310 | 299 | 322 | 272 |
| 1800 | 1980 | 289 | 259 | 248 | 271 | 226 |
| 2000 | 2200 | 244 | 215 | 206 | 228 | 187 |

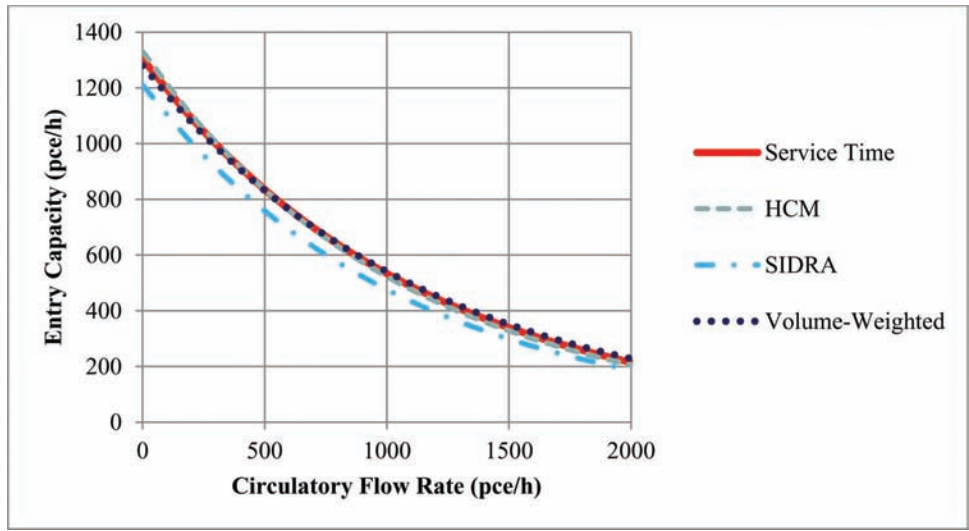


Figure 6.4 Effect of 10% heavy vehicles on the entry capacity based on service time, HCM, SIDRA, and volume-weighted methods for Indiana conditions.

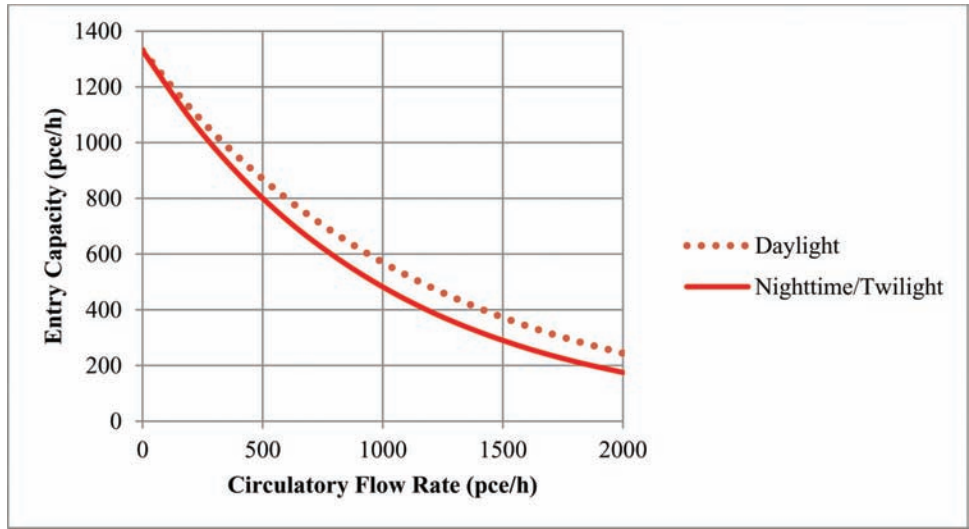


Figure 6.5 Effect of nighttime/twilight on the entry capacity for Indiana conditions.

over estimating the entry capacity at low circulating traffic. The SIDRA method produces the capacity estimates lower than the other methods. Evaluation of the reliability of these methods is recommended.

The effect of nighttime/twilight condition (in the presence of street lighting) indicated additional capacity reduction caused by a 0.6 sec longer critical headway than in daylight conditions, which was possibly due to poor visibility and the glare effect, which can adversely affect driver perception, resulting in longer critical headways. The reduction in capacity due to nighttime/twilight conditions is shown in Figure 6.5.

Moreover, the number of rejected headways more than one, as an indicator variable, was statistically significant. The parameter sign was positive as expected, which implied that drivers who inspect the available shorter headways adapt to the existing condition and finally accept a shorter headway. Drivers in this situation accepted 0.5 sec shorter critical headways, on average, as indicated by the results.

On the other hand, the effect of the right turning maneuvers on the critical headway was not statistically different from other turns, and the effect of the right lane was not statistically different from the left lane.

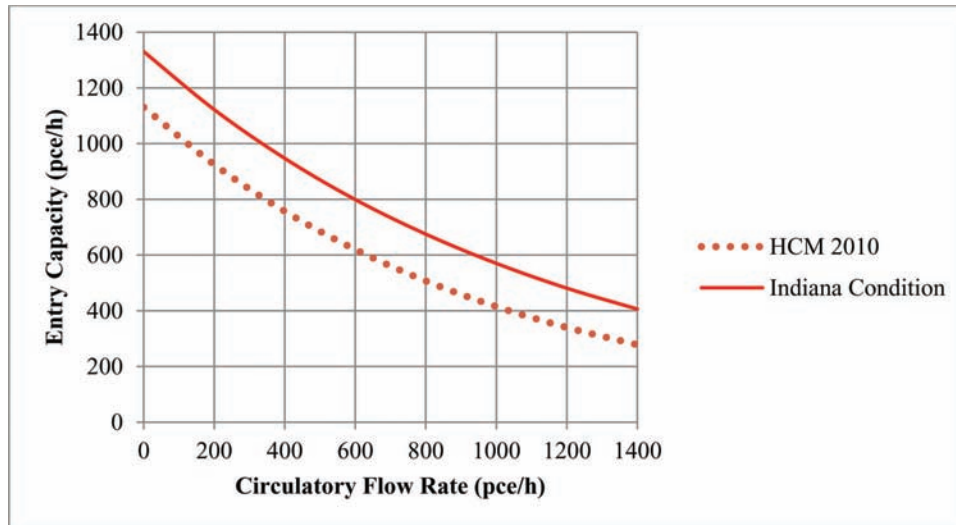


Figure 6.6 Entry capacity of single-lane roundabouts for Indiana conditions vs. that of the HCM 2010.

However, drivers accepted shorter headways when turning right or when entering the roundabout from the right lane than other drivers. This result may be attributed to the shorter paths across the conflict areas on roundabouts followed by these drivers than by other drivers. This may lead to higher confidence and to accepting shorter headways.

6.4.2 Indiana Conditions vs. HCM 2010

The mean critical headway for the studied single-lane roundabout was estimated 4.4 sec, which is 0.7 sec shorter than the NCHRP Report 572 average findings of 5.1 sec for single-lane roundabouts. In a separate calculation, the follow-up headway was estimated 2.7 sec, which is 0.5 sec smaller. Since functions *A* and *B* of the HCM capacity model depend upon the gap-acceptance parameters, the new values that reflect the local condition were 1,330 (as opposed to 1,130) and 0.00085 (as opposed to 0.001), respectively. The calibrated model for single-lane roundabouts on state roads in urban areas, based on the case study, is shown in Equation 6.4.

$$C_e = 1,330e^{(-0.85 \times 10^{-3})v_c} \quad (6.4)$$

The effects of the estimated gap-acceptance parameters on the entry capacity for different circulating traffic conditions are shown in Figure 6.6. For comparison purposes, the HCM entry capacity for single-lane roundabouts is also illustrated in the same figure. In the ideal situation when there is no conflicting traffic, the saturation flow rate (the maximum traffic flow a lane can serve in one hour) depends upon the follow-up headway only and is 1,330 pce/h for the local condition, which is 200 pce/h higher (18% increase) than that of the HCM for roundabouts. At heavy traffic (e.g.,

1,400 veh/h) this difference is approximately 130 pce/h (46% increase). The difference in capacities can be averaged as 30% increase for local conditions. Generally, this implies that drivers are more accustomed to roundabouts in urban areas and accept smaller headways, which improves the capacity.

On the other hand, the critical headway on dual-lane roundabouts in rural areas was estimated 5.0 sec, on average. The estimated critical headway is larger than the average critical headways for the left and right lanes reported in NCHRP Report 572. In contrast to the NCHRP 572 findings, the critical headway in the right lane compared to the left lane was not statistically significant. On rural high-speed roads, drivers experience lower delays than on low-speed urban roads due to fewer traffic control features (e.g., intersections), which implies that drivers reject longer headways. This behavior may become more aggressive when rural roads start experiencing longer delays. On the other hand, the follow-up headway was estimated 2.7 sec, on average, which is 0.5 sec shorter than the NCHRP Report 572 findings for dual-lane roundabouts. The calibrated equation, based on the new estimated gap-acceptance parameters, for dual-lane roundabout in rural areas is shown in Equation 6.5.

$$C_e = 1,330e^{(-1.0 \times 10^{-3})v_c} \quad (6.5)$$

The difference in the entry capacity is shown in Figure 6.7 for a range of circulating traffic. As can be seen, the entry capacity is higher (10% increase, on average) for light circulating traffic (up to 500 pce/h) and lower (15% decrease, on average) for medium to heavy circulating traffic (500–2,000 pce/h), compared to the left lane calculated capacity from the HCM equation. The implication is that drivers behave

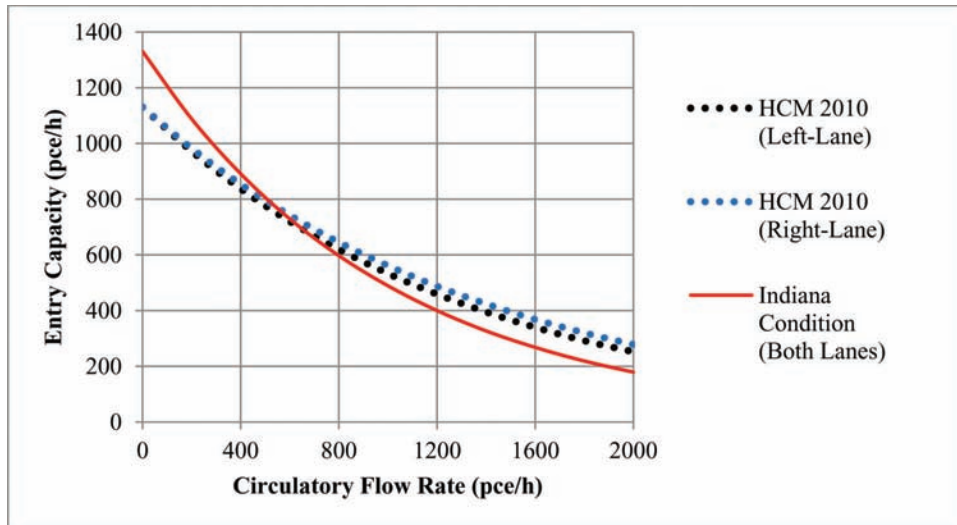


Figure 6.7 Entry capacity of dual-lane roundabouts for Indiana conditions vs. that of the HCM 2010.

TABLE 6.7
Estimated Critical Headways.

| Model | Assumptions | Distribution | Critical | | |
|--------|------------------------------|--------------|-------------|---------------|--------------------|
| | | | Sample Size | Headway (sec) | Standard Deviation |
| Probit | Inconsistent driver behavior | Normal | 1149* | 4.478 | 0.958 |
| MLM | Consistent driver behavior | Log-normal | 580 | 4.175 | 0.796 |

*The sample includes all rejected headways as opposed to the largest ones.

differently on roundabouts on high-speed approaches; this was expected as roundabout is relatively a new traffic control feature on high-speed roads.

The calibrated capacity equations for both single-lane and dual-lane roundabouts are helpful for capacity estimation of Indiana roundabouts on state roads.

6.4.3 Model Evaluation

The estimated gap-acceptance parameters from three hours of traffic operations on a single-lane roundabout were used for the simulation purpose. The results of the

estimated critical headways, based on different methods, are shown in Table 6.7. The average follow-up time was 2.7 sec with a standard deviation of 1.0 sec.

The gap-acceptance behaviors of the same drivers were simulated based on the estimated parameters in such a way that random critical headways were generated based on random probabilities (between 0 and 1) and the estimated mean and standard deviation, which was consistent with the assumptions of the used methods. For consistent behavior one critical headway was generated for one approaching driver while for inconsistent behavior as many critical headways as the number of decisions of the same approaching driver were generated. It is worth mentioning that unlike previous studies reviewed in literature, traffic was not generated on the entering or circulation roadways, rather the behavior of the actual drivers were simulated.

The delay at the first position of the queue was set as a criterion. The estimated delay based on simulation was compared to the measured delay from the actual observations. The results are shown in Table 6.8. The simulation results indicated very close average delays between the scenarios. The *t*-statistic test showed that the differences were not statistically significant among the simulated scenarios as well as with the actual one. Nevertheless, the average delays resulted from the

TABLE 6.8
Simulation Results to Evaluate Different Methodological Assumptions for Critical Headway Estimation.

| Scenario | Delay at the First Position of the Queue (sec/veh) | |
|--|--|--------------------|
| | Average | Standard Deviation |
| Actual | 3.364 | 7.471 |
| Inconsistent driver behavior and normal distribution of critical headways (Probit) | 3.419 | 6.295 |
| Consistent driver behavior and log-normal distribution of critical headways (MLM) | 3.296 | 7.530 |

Probit estimated critical headways are slightly on the conservative side and the assumption of inconsistent driver behavior seems to be more realistic than the assumption of fully consistent behavior.

Furthermore, the difference in the results when all the rejected headways were used, were rather limited, comparable to the case with only the largest rejected headway (4.424 sec as opposed to 4.251 sec), the estimated models are shown in Appendix C (Tables C-3 and C-4). Using all the rejected headways corresponds to the assumption of the lack of driver consistency in rejecting headways while selecting the largest value is equivalent to the assumption of full consistency. Thus, the assumption of inconsistent driver behavior allows using all the data collected which contributes to a more confident estimation of the critical headways and to a more adequate model that is not contradicted by the observable data.

To summarize the above discussions, a number of factors, including vehicle type and lighting condition, influence driver gap-acceptance behavior on roundabouts, which in turn affect the capacity. Ignoring such factors may lead to inaccurate capacity estimation and less of an understanding of roundabout operational performance. Furthermore, using the default HCM 2010 capacity equations for roundabouts without calibrating to local conditions may over- or under-estimate the capacity for these conditions. Furthermore, a realistic and efficient estimation method of the gap acceptance parameters is important; the assumption of inconsistent driver behavior may be expected to result in more accurate estimations.

7. CONCLUSIONS

7.1 Safety

Due to their safety and capacity benefits demonstrated in past research, roundabouts are likely to continue their emergence as a choice alternative intersection. With regards to safety, the literature suggests that roundabouts are highly effective in reducing severe and fatal accidents. Roundabout construction on high-speed roads has recently commenced. Although crash statistics show consistency with those on low-speed roads in reducing the most severe accidents, these roundabouts bring new challenges on how to safely accommodate the considerable heavy vehicle traffic. Experience from the United States and other countries show that the rollover risk of heavy vehicles should be considered in roundabout design. Some roundabout geometric features linked with rollover have been identified, but there are still questions that need to be answered before efficient safety countermeasures can be determined. Each of the safety-related chapters in this report describes different design, driver, and environmental scenarios that are needed for a more complete understanding of roundabout safety performance.

This study provided primary contributions and a foundation for future research. First, it developed a

more generalized model for heavy vehicle rollover, accounting for complex paths and tilt experienced by semis and other heavy vehicles in roundabouts. The presented study applied the advanced rollover propensity model to data collected in the field. The established methodology can be readily used for investigating other types of road curves if sufficient geometric and motion data are available. Finally, it evaluates the true safety margin of actual truck drivers by indicating the proximity to rollover without disturbing their behavior.

Although this study did find a difference between the roundabouts on low and high-speed roads in terms of the proximity to rollover in the circulation, the difference could not be connected to the driver speeds on the approach roads. Drivers on the high-speed approach began their deceleration earlier and had similar speed profiles to drivers on the low-speed approach close to the roundabout. The low-speed, single-lane roundabout had a smaller average minimum Δv_c on the circulatory roadway, 2.6–3.4 mph lower depending on the assumed trailer loading. This single-lane roundabout limited driver path selection to a greater extent than the high-speed, two-lane roundabout. The single-lane roundabout tended to confine drivers' speed choice which might have been the main cause of the higher rollover risk in the circulation compared to the two-lane roundabout. The wider circulatory roadway of the two-lane roundabout appeared to reduce this concern for drivers by decreasing their propensity for overturning.

It was confirmed that inward circulatory superelevation gives a statistically significant, higher Δv than the typically used outward design. However, the safety effect is too small to provide support for the inward design given its other shortcomings, such as a sudden change in cross slope between the roundabout approach curve and circulation and difficulties in inward drainage. These results may point toward continuing the design practice of outward circulatory superelevation.

The report indicates that aggressive driving manifested through high speed far from the roundabout does not imply a larger risk of rollover in the roundabout.

The results suggest that the percentage of drivers exceeding the comfortable deceleration rate immediately prior to the roundabout approach curve was greater on the high-speed road as compared to the low-speed road. Furthermore, it was found that a greater deceleration rate used by drivers immediately before the approach curve could be connected to an increased rollover propensity on the approach curve. It should be noted that this rollover margin was still large. However, it may become a greater safety issue if the roundabout approach curve is designed with too small of a radius.

The model's application improved the understanding light conditions have on the rollover threshold of heavy vehicles. Under nighttime conditions, truck drivers were found to behave more conservatively on the roundabout approach, overcompensating during the night-

time conditions. While still slightly more conservative under nighttime conditions, the closeness to rollover was more on the roundabout circulatory roadway. This suggests some behavioral adjustment and more confident behavior after entering the roundabout.

The literature review and the inspection of crash statistics gave additional insight into the rollover issue leading to proposing additional design improvements for consideration. To prevent rollover after a truck goes over the apron, the apron should be designed as easily mountable, or better, flashed and marked with the texture and color different from the pavement in the circulatory roadway. Drivers of heavy vehicles need to be better informed and trained to maneuver a roundabout without increasing the risk of rollover.

Finally, although not explicitly considered in the analysis due to their relative scarcity, trucks that are overweight (in excess of 80,000 lb gross vehicle weight) are expected to have a smaller margin to rollover Δv than those not overweight. This is partially due to the increased load weight, but more attributable to the increased center of mass height. With the appropriate truck loading information, the developed rollover model is applicable for studying overweight vehicles.

In summary, the detailed and careful analysis of truck drivers' behavior on selected Indiana roundabouts did not detect any excessive rollover risk on the studied roundabouts built on high-speed roads.

The effect of the circulatory roadway sloped inward was too small to justify the increased design and drainage difficulties expected when applying the inward superelevation. Strong braking in close vicinity of the approach curve was associated with a higher rollover risk, but still presented a large rollover margin of safety. Truck drivers' high speeds far in advance to the roundabout was not associated with any considerable increase in the rollover propensity at the roundabout. It was found, however, that the two-lane circulatory roadway allowed truck drivers to compensate for higher speeds with flatter paths. This compensation slightly reduces the rollover propensity. The night conditions increased cautious driving associated with a lower rollover propensity on the studied roundabout. The findings related to heavy vehicle safety do not provide the basis for recommending changes in the current Indiana design policy for roundabouts. All these findings were obtained for the period when roundabouts on high-speed roads were still infrequent. It is expected that with proliferation of roundabouts in Indiana, the drivers' familiarity with roundabouts will increase. A change in the risk perception and in the behavior may occur. Future studies should be conducted with more roundabouts as they continue to emerge.

7.2 Capacity

Previous studies on roundabouts mainly focused on mean critical headway and follow-up headway estimation. Limited research was found in the literature review

that investigated the effects of heavy vehicles and other factors influencing these parameters. Furthermore, most of the studies were on roundabouts in urban/suburban areas. The motivation for the present research was to investigate the effects of heavy vehicles, along with the area type and nighttime/twilight conditions, on the critical headway and follow-up headway of drivers maneuvering roundabouts on high-speed roads.

This report revealed that heavy vehicles increased the critical headway, and in turn reduced the entry capacity of roundabouts. Drivers of heavy vehicles, on average, accepted a 1.1 sec longer critical headway than drivers of passenger cars. The effects of nighttime/twilight conditions indicated additional capacity reduction caused by a 0.6 sec longer critical headway compared to daylight conditions. Likewise, drivers on dual-lane roundabouts in rural areas accepted a 0.6 sec longer critical headway than drivers on single-lane roundabouts in urban areas. Furthermore, the number of rejected headways more than one, as an indicator variable, was found statistically significant with a positive sign. Contrary to some previous research results, including NCHRP Report 572, the difference between the critical headways for the left and right lanes on dual-lane roundabouts was not statistically significant. Also, the difference in critical headways for the right turning movement compared to other turns (through, left and U-turn) was not statistically significant.

Moreover, it was determined that the gap-acceptance parameters for a single-lane roundabout on a low-speed state road were less than those of the National Cooperative Highway Research Program (NCHRP) Report 572 average estimated values—which are currently incorporated into Highway Capacity Manual (HCM; TRB, 2010), resulting on average in 30% higher capacity for Indiana conditions. In contrast, the estimated critical headway was larger for dual-lane roundabouts on high-speed state roads, resulting in 15% reduced capacity (for medium to high circulatory traffic volumes) for Indiana conditions.

The MLM (Troutbeck) method is widely used for estimating the mean and variance of the critical headway. However, this method does not account for the fact that driver behavior may be inconsistent (i.e., drivers may accept shorter gaps than the largest associated rejected gaps). Furthermore, the MLM method was not designed to determine the influence of other factors in the critical headway estimation. Therefore, the concept of standard binary Probit method was used in this report in order to relax some of the MLM assumptions. In addition, the observed driver behaviors (from video records) and the findings from simulations revealed that the assumption of inconsistent driver behavior in gap-acceptance analysis is valid and leads to more reasonable estimations.

Consequently, the critical headway estimates were obtained with all the rejected headways using the Probit model. The obtained estimates of the critical headway

were only slightly different from the estimate obtained with the MLM method when only the largest rejected headway for each driver were used. Nonetheless, inclusion of full information (all rejected headways) is recommended to account for inconsistent driver behavior and to obtain more reliable estimates.

The findings of this report are intended to improve capacity estimation for the roundabouts planned on Indiana state roads. The HCM 2010 capacity equations were updated with the new estimated gap-acceptance parameters for Indiana. These new values may be used by INDOT designers and traffic engineers. The findings contribute to a better understanding of roundabout capacity factors.

The research findings may be helpful in improving capacity estimation for Indiana roundabouts located on high-speed state roads. Studying more roundabouts on high-speed roads, particularly, in nighttime conditions is recommended. Furthermore, roundabouts still may be new to many drivers so repeating similar studies in the future is needed to update the knowledge after more drivers have adjusted to this relatively new design and to more frequent delays on state roads.

Since this report covered a limited number of sites in the state of Indiana, the results need to be improved by studying more sites around the country in order to generalize the effects of the studied conditions on the capacity of roundabouts built on high-speed roads.

The findings of this report are based on low and medium traffic volumes presently observed on high-speed rural and suburban roads. Heavy traffic flow may affect driver behavior; therefore, studying such roundabouts in heavier traffic conditions might improve the results.

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APPENDIX A: DESCRIPTIONS OF HEAVY VEHICLE ROLLOVERS

US-400 and K-47 near Fredonia

Date: 12/29/2010 **Light:** Daylight **Weather:** Fog

Narrative: "Vehicle was eastbound on U400 approaching the roundabout at the K47 junction. Vehicle's speed was too fast approaching the roundabout and overturned while negotiating the curves prior to it."

Date: 1/10/2011 **Light:** Dark, with street lights on **Weather:** Snow

Narrative: "D1 was traveling east on U-400. D1 was traveling too fast for the road conditions. D1 lost control of V1 as he entered curve prior to round-about. V1 slid sideways until it struck curb and rolled over onto its driverside and spun around facing north or west U-400."

Date: 3/5/2012 **Light:** Daylight **Weather:** No adverse weather

Narrative: "Vehicle was west bound on U400 approaching the round about at K47. Vehicle entered the round about with too much speed to safely negotiate the round about and rolled onto its left side."

Date: 06/16/2013 **Light:** Daylight **Weather:** No adverse weather

Narrative: "Vehicle 1 was traveling east on U400. Vehicle 1 came into roundabout too fast and overturned on its passenger side."



Figure A.1 US-400 and K-47 roundabout near Fredonia, KS (Google Earth).

US-50 and I-35 Access Road at Emporia

Date: 07/23/2011 **Light:** Dark, with street lights on **Weather:** No adverse weather

Narrative: "V1 was going east on US Hwy 50. V2 was going east on US Hwy 50 ahead of V1. V2 entered the roundabout. V1 attempted to turn into the roundabout, turned over on its side, and slid across the roadway hitting V2."



Figure A.2 US-50 and I-35 Access Road roundabout at Emporia, KS (Bing Maps).

US-50, US-77, and 8th Street near Florence

Date: 1/2/2007 **Light:** Daylight **Weather:** No adverse weather

Narrative: "V1 approached the roundabout on US 50 Highway East. V1 failed to slow down. V1 traveled through the roundabout. DV1 over corrected V1, units three and four over turned. Damaging Units 2-4 and two State traffic signs."

Date: 5/4/2013 **Light:** Dark, with street lights on **Weather:** Rain

Narrative: "Unit 1 raveling entered roundabout speed to great hit center of intersection veered right crossed lane hit other curb overturned landing in north ditch."



Figure A.3 US-50, US-77, and 8th Street Roundabout near Florence, KS (Google Earth).

US-59 and US-169 near Garnett

Date: 8/29/2011 **Light:** Daylight **Weather:** No adverse weather

Narrative: "V1 was S/B on U-169. V1 entered the roundabout and turned over as traveling through the roundabout."



Figure A.4 US-59 and US-169 Roundabout near Garnett, KS (Google Earth).

56th Avenue and Plum Street near Hutchinson

Date: 7/15/2010 **Light:** Daylight **Weather:** No adverse weather

Narrative: "U1 was traveling through the roundabout at 56th/Plum. The trailer wheels went up onto the brick curb of the roundabout and caused the cargo in the trailer to shift and the tractor trailer overturned onto its right side.



Figure A.5 56th Avenue and Plum Street near Hutchinson (Google Earth).

APPENDIX B: DESCRIPTIVE STATISTICS

This appendix presents some descriptive statistics of the observed headways.

Basic Statistics

The observed follow-up headways varied from 1.0 sec to 5.0 sec for all the studied roundabouts. (See Table B.1.)

Rejected/Accepted Headway Distributions

Utilizing EasyFit tool, the probability density functions (PDFs) of the measured rejected and accepted headways, for the single-lane roundabout, are shown in Figure B.1. The best fit, among over sixty distribution types programmed in the EasyFit tool, was the Pearson 5 for rejected headways and Burr for accepted headways. This was not the case for all the studied roundabouts. Therefore, it is implied that the observed rejected/accepted

headways distribution is not reasonable to use as a base for the latent critical headway distribution.

TABLE B.1
Observed Maximum Rejected and Minimum Accepted Headways.

| Roundabout | Max Rejected Headway (sec) | Min Accepted Headway (sec) |
|---|----------------------------|----------------------------|
| SR 25: Old SR 25, Lafayette | 6.33 | 4.87 |
| SR 32/38: Union Chapel Road, Noblesville | 6.14 | 2.29 |
| SR 32/38: Promise Road, Noblesville | 6.27 | 4.26 |
| Indiana 130: LaPorte Ave-N. Sturdy Road, Valparaiso | 7.31 | 2.57 |

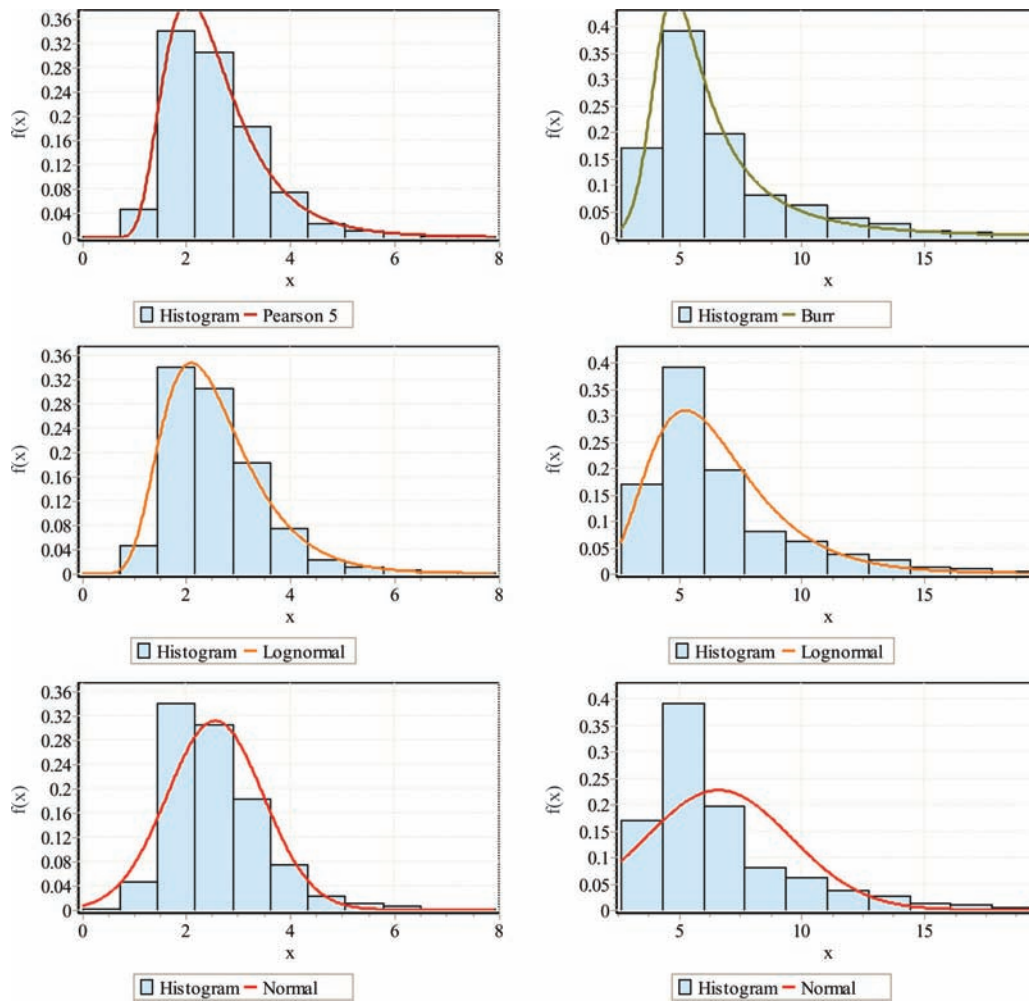


Figure B.1 Probability density functions (PDFs) of the measured rejected and accepted headways for the studied single-lane roundabout (left: rejected headways; right: accepted headways).

APPENDIX C: INTERMEDIATE RESULTS FROM SAS BINARY PROBIT MODELS

This appendix presents the results of several statistical models that have been pointed to in Chapter 6. The estimated mean and

standard deviation of the critical headway, based on the model shown in Table C.3, are 4.424 sec and 0.943 sec, respectively. The estimated mean and standard deviation of the critical headway, based on the model shown in Table C.4, are 4.251 sec and 1.101 sec, respectively.

TABLE C.1
Binary Probit Model for Critical Headways (NRH is Not Considered).

| Variable | Parameter Estimate | t-value |
|---|--------------------|---------|
| Constant (intercept) | -4.480 | -28.59 |
| Measured headway | 1.006 | 26.59 |
| Dual-lane in rural area | -0.568 | -4.03 |
| Heavy vehicles (trucks and buses) | -1.091 | -3.92 |
| Nighttime/twilight (in the presence of street lighting) | -1.198 | -2.94 |
| Number of observations | 2,894 | |
| Maximum likelihood at convergence | -529.425 | |

TABLE C.2
Estimated Critical Headways for the Studied Conditions (Based on Table C.1).

| Condition | Sample Size | Critical Headway (sec) |
|--|-------------|------------------------|
| The base-case condition (single-lane, urban area, passenger car, daylight) | 2121 | 4.4 (1.0)* |
| Dual-lane in rural area | 544 | 5.0 (1.0) |
| Heavy vehicles (trucks and buses) | 108 | 5.5 (1.0) |
| Nighttime/twilight (in the presence of street lighting) | 121 | 5.6 (1.0) |

*Standard deviations in parentheses.

TABLE C.3
Binary Probit Model for Critical Headways (Including All Rejected Headways).

| Variable | Parameter Estimate | t-value |
|-----------------------------------|--------------------|---------|
| Constant (intercept) | -4.690 | -25.97 |
| Measured headway | 1.060 | 24.19 |
| Number of observations | 2,121 | |
| Maximum likelihood at convergence | -434.260 | |

TABLE C.4
Binary Probit Model for Critical Headways (Only the Largest Rejected Headways).

| Variable | Parameter Estimate | t-value |
|-----------------------------------|--------------------|---------|
| Constant (intercept) | -3.860 | -19.18 |
| Measured headway | 0.908 | 19.25 |
| Number of observations | 1,152 | |
| Maximum likelihood at convergence | -381.160 | |

About the Joint Transportation Research Program (JTRP)

On March 11, 1937, the Indiana Legislature passed an act which authorized the Indiana State Highway Commission to cooperate with and assist Purdue University in developing the best methods of improving and maintaining the highways of the state and the respective counties thereof. That collaborative effort was called the Joint Highway Research Project (JHRP). In 1997 the collaborative venture was renamed as the Joint Transportation Research Program (JTRP) to reflect the state and national efforts to integrate the management and operation of various transportation modes.

The first studies of JHRP were concerned with Test Road No. 1—evaluation of the weathering characteristics of stabilized materials. After World War II, the JHRP program grew substantially and was regularly producing technical reports. Over 1,500 technical reports are now available, published as part of the JHRP and subsequently JTRP collaborative venture between Purdue University and what is now the Indiana Department of Transportation.

Free online access to all reports is provided through a unique collaboration between JTRP and Purdue Libraries. These are available at: <http://docs.lib.purdue.edu/jtrp>

Further information about JTRP and its current research program is available at: <http://www.purdue.edu/jtrp>

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