NCHRP REPORT 572

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Roundabouts in the United States

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

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FOREWORD

By B. Ray Derr
Staff Officer
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Based on a comprehensive evaluation of roundabouts in the United States, this report presents methods of estimating the safety and operational impacts of roundabouts and updates design criteria for them. The report will be useful to geometric designers and traffic engineers who are considering improvements to an intersection. Presentation materials that may be helpful in public meetings and similar forums are available on the TRB website (http://www.trb.org/news/blurb_detail.asp?id=7086).

Although traffic circles have been used in the United States since 1905, their use has been limited since the 1950s because many were found to work neither efficiently nor safely. The modern roundabout was developed in the United Kingdom in the 1960s to address these problems. Two key characteristics of the modern roundabout are (1) entering traffic that yields to circulating traffic and (2) geometric constraints that slow entering vehicles. Many studies have shown that modern roundabouts (hereafter referred to as roundabouts) are safe and effective, and they are now widely used internationally.

Because roundabout design is relatively new to the United States, there has been some reluctance to use them. Perceived differences in driver behavior raise questions about how appropriate some international research and practices are for the United States. Therefore, additional information on the safety and operation of roundabouts in the United States will be very helpful to planners and designers in determining where roundabouts would reduce intersection crashes and congestion and in refining the design criteria currently being used. These design refinements can be particularly important for bicyclists and pedestrians using the intersection.

Under NCHRP Project 3-65, Kittelson & Associates, Inc. and their subcontractors reviewed existing safety and operational models. After compiling a comprehensive inventory of roundabouts in the United States, they traveled to several representative ones to gather geometric, operational, and safety data. Particular emphasis was placed on collecting data at roundabouts with significant pedestrian and bicycle volumes. They then evaluated the different analytical models to determine how well they replicate U.S. experience. The best models were then refined. During the course of the project, the research team also gathered information on transportation agencies' experiences with different design configurations.

NCHRP Web-Only Document 94 (http://www.trb.org/news/blurb_detail.asp?id=7274) contains the appendixes to this report and includes detailed reviews of the literature on safety performance and operational models, the master inventory of U.S. roundabouts assembled for this project, and the results of the statistical testing of various models.

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The authors appreciate the many contributions of data that support the findings of this project. People from agencies all across the United States, too numerous to mention individually, supplied crash data, design plans, and other information, as well as facilitated our field visits during scouting and data collection. The authors also thank Mr. Srinivas Mandavilli of Kansas State University for being available on short notice to assist with data collection in Kansas and Mr. Michael Wallstedt of TranSystems, Inc. and Mr. Leif Ourston of Ourston Roundabout Engineering for providing supplementary video footage.

The authors especially thank the NCHRP 3-65 panel members for long and diligent service in reviewing and guiding this project and greatly appreciate the guidance and patience from the Senior Program Officer of the National Cooperative Highway Research Program.

SUMMARY

Roundabouts in the United States

Based on the findings of this study, roundabouts appear to be successful in a wide variety of environments in the United States. The following sections summarize the major conclusions from this study.

Safety Performance

In general, roundabouts have improved both overall crash rates and, particularly, injury crash rates in a wide range of settings (urban, suburban, and rural) for all previous forms of traffic control except for all-way stop control, for which no statistically significant difference could be found. In addition, single-lane roundabouts have better safety performance than multilane roundabouts. The safety performance of multilane roundabouts appears to be especially sensitive to design details.

This study produced a number of major safety findings:

- Intersection-level crash prediction models for the prediction of the overall safety performance of the intersection. These models relate the crash prediction to the number of lanes, number of legs, and the average annual daily traffic.
- Approach-level crash prediction models that relate common types of crashes (e.g., exiting-circulating crashes) to average annual daily traffic volumes and key geometric parameters that were demonstrated to influence the prediction.
- An updated comparison of the performance of roundabouts to other forms of traffic control, disaggregated to a greater extent than any previous study of U.S. roundabouts.

Operational Performance

Currently, drivers in the United States appear to use roundabouts less efficiently than models suggest is the case in other countries around the world. In addition, geometry in the aggregate sense—number of lanes—has a clear effect on the capacity of a roundabout entry; however, the fine details of geometric design—lane width, for example—appear to be secondary and less significant than variations in driver behavior at a given site and between sites.

The following model is recommended for the entry capacity at single-lane roundabouts:

$$c = 1130 \cdot \exp(-0.0010 \cdot v_c) \tag{S-1}$$

where

c = entry capacity (passenger car units [pcu]/h)

v_c = conflicting flow (pcu/h)

Because driver behavior appears to be the largest variable affecting roundabout performance, calibration of the models to account for local driver behavior and changes in driver experience over time is highly recommended to produce accurate capacity estimates. The exponential model parameters can be calibrated using locally measured parameters as follows:

$$c = A \cdot \exp(-B \cdot v_c) \tag{S-2}$$

where

c = entry capacity (pcu/h)
$A = 3600/t_f$
$B = (t_c - t_f/2)/3600$
v_c = conflicting flow (pcu/h)
t_f = follow-up headway (s)
t_c = critical headway (s)

The recommended capacity model for the critical lane of a multilane entry into a two-lane circulatory roadway is as follows:

$$c_{crit} = 1130 \cdot \exp(-0.0007 \cdot v_c) \tag{S-3}$$

where

c_{crit} = entry capacity of critical lane (pcu/h)
v_c = conflicting flow (pcu/h)

The recommended control delay model follows:

$$d = \frac{3600}{c} + 900T\left[\frac{v}{c} - 1 + \sqrt{\left(\frac{v}{c} - 1\right)^2 + \frac{\left(\frac{3600}{c}\right)\frac{v}{c}}{450T}}\right] \tag{S-4}$$

where

d = average control delay (s/veh)
c = capacity of subject lane (veh/h)
T = time period (h: T = 1 for 1-h analysis, T = 0.25 for 15-min analysis)
v = flow in subject lane (veh/h)

The recommended level of service (LOS) criteria are the same as those currently used for unsignalized intersections. The LOS for a roundabout is determined by the computed or measured control delay for each lane. The LOS is not defined for the intersection as a whole.

These models have been incorporated into an initial draft procedure for the *Highway Capacity Manual*, which the TRB Committee on Highway Capacity and Quality of Service will continue to revise until its eventual adoption.

Geometric Design

This study produced a number of major geometric design findings:

- The application of acceleration and deceleration effects appears to significantly improve the ability to predict 85th-percentile speeds entering and exiting a roundabout.
- The combination of the extensive field observations of critical gap and the revised speed predictions may be used to refine the current intersection sight distance procedure presented

in FHWA's *Roundabouts: An Informational Guide*. These findings should be considered interim until a more comprehensive study of sight distance needs at roundabouts can be completed.
- Anecdotal evidence suggests the importance of considering design details in multilane roundabout design, including vehicle path alignment, lane widths, and positive guidance to drivers through the use of lane markings.

Pedestrian and Bicyclist Observations

This study produced a number of findings regarding pedestrian and bicyclist behavior at roundabouts:

- This research did not find any substantial safety problems for non-motorists at roundabouts, as indicated by few crashes being reported in detailed crash reports. In addition, no crashes and a very small number of conflicts were observed from video recordings of interactions between non-motorists and motorists. Because exposure data were not available from before a roundabout was present, it is unknown whether pedestrians have altered their travel patterns because of the presence of a roundabout.
- The ability of pedestrians and bicyclists to use the roundabout may be compromised if use of the roundabout by all modes and their subsequent interactions are greater than studied herein or if such interactions increase over time (i.e., as vehicle traffic and/or pedestrian traffic increases).
- An emphasis needs to be placed on designing exit lanes to improve upon the behaviors of both motorists and pedestrians.
- Multilane roundabouts may require additional measures to improve upon the behaviors of motorists, pedestrians, and bicyclists.

CHAPTER 1

Introduction and Research Approach

This report summarizes the findings of NCHRP 3-65, "Applying Roundabouts in the United States." The intended audience for this report is researchers, practitioners, and policy makers who establish federal, state, and local guidelines for roundabouts. Although the content of this document is directly relevant to practitioners, the document is not organized as a guide for easy practitioner use. Once these findings are incorporated into the next edition of FHWA's *Roundabouts: An Informational Guide* (*1*) and other key guidance documents, they should be more accessible to practitioners.

This introductory chapter presents the problem statement and research objective, the scope of study, research approach, and a summary of literature review conducted for this project.

Problem Statement and Research Objective

Although traffic circles have been used in the United States since 1905, their use has been limited since the 1950s because the designs of that era were found to work neither efficiently nor safely. The modern roundabout was developed in the United Kingdom (UK) in the 1960s to address these problems. Two key characteristics of the modern roundabout include (1) a requirement for entering traffic to yield to circulating traffic and (2) geometric constraints that slow entering vehicles. Many studies have shown that modern roundabouts (hereafter referred to simply as roundabouts) can be safe and effective, and they are now widely used internationally. Use in the United States began in 1990 and has been increasing exponentially since that time.

Due to this increased interest in roundabouts, continued demand exists for more information regarding appropriate physical locations, design parameters, and their performance relative to alternative control schemes, with a particular need for that information to be based on U.S. performance rather than simple continued reliance on international experience. The lack of comprehensive and objective U.S. field data on safety and operational performance and design of roundabouts has contributed to this demand for information, as perceived differences in driver behavior raise questions about how appropriate some international research and practices are for the United States. Therefore, additional information on the safety and operation of roundabouts in the United States will be very helpful to planners and designers in determining where roundabouts would reduce intersection crashes and congestion and in refining the design criteria currently being used.

NCHRP and FHWA have identified the need to develop tools based on actual U.S. roundabout performance, rather than using foreign procedures as surrogates. Hence, the primary objective of this research is to produce a set of operational, safety, and design tools, calibrated to U.S. roundabout field data. These tools will enable a person who is already competent in analysis or geometric design of typical at-grade intersections to be able to specify a roundabout that is safe and performs well.

Scope of Study

The scope of this study includes the development of the following work products:

- An updated site inventory of known roundabouts that is accessible to the transportation profession
- A comprehensive database of safety, operational, and design data of selected existing roundabouts for use in future research
- Planning-level safety prediction models to predict the overall safety performance of roundabouts
- Design-level safety prediction models for individual roundabout approaches
- An expanded comparison of safety performance before and after installation of a roundabout
- An updated operational analysis procedure for the *Highway Capacity Manual* (HCM) (*2*), including capacity,

delay, and queue estimates for single-lane and multilane roundabouts
- New speed prediction tools for use in design development
- A comprehensive study of pedestrian and bicyclist behavior at roundabouts
- Updated design guidance incorporating the results from the various studies identified above.

It became clear during the early stages of this research effort that the proposed scope of work and associated data collection effort would be insufficient to address issues related to the accommodation of visually impaired pedestrians at roundabouts. As a result, issues specifically related to visually impaired pedestrians were removed from this scope and combined with channelized right turns at conventional intersections as part of a new problem statement spawned from this project: NCHRP 3-78, "Crossing Solutions for Visually Impaired Pedestrians at Roundabouts and Channelized Right Turns."

Research Approach

The detailed approaches for each of the major components of this research are described in the following sections.

Summarize Existing Relationships

Existing models in use around the world for roundabout safety and operational analysis were described, analyzed, and critiqued to understand the current state of practice in roundabout safety and operational modeling. This literature review is presented later in this chapter.

Site Inventory and Data Collection

A major element of the study included updating and expanding the inventory of U.S. roundabouts compiled during recent research and volunteer efforts and making the inventory available to transportation professionals. The products of this task include an updated database that includes information and data on as many roundabouts in the United States that the team could locate and retrieve information about, including components that are available on line.

At selected sites, the research team collected and summarized extensive data on operational performance, safety performance, geometric parameters, and speeds. Specific data collection methods included assembly of crash reports, crash summaries, and plans; extensive video recording during peak and off-peak periods; and spot speed measurements using radar guns. These methods are described in detail in subsequent chapters of this report.

Operational Model Development

Operational model development included the following tasks:

- **Evaluation of existing models and software.** This task consisted of comparing the field data collected for each roundabout to the predictions from a wide range of existing roundabout capacity models, plus two major software implementations in use in the United States (RODEL and aaSIDRA). The evaluation focused primarily on the ability of each model to predict capacities, delays, and/or queues under the geometric and traffic flow conditions observed at U.S. roundabouts.
- **Development of two operational models that attempt to best fit the U.S. data and explain the performance of U.S. roundabouts.** The capacity models considered comprise the full range of potential formulations, including empirical regression and analytical formulations (gap acceptance). Delay and queuing models are based on those currently in use in the HCM for predicting performance at other unsignalized intersections.
- **Development of a draft revised HCM procedure that incorporates the findings from this research.**

Safety Model Development

Unlike operational model development in the United States, where the HCM has been a definitive reference for more than 50 years, safety model development in the United States is in its infancy, with the first edition of the *Highway Safety Manual* still in development at the time of this research. Other countries have successfully developed safety models, but it has been unclear if these models are directly transferable to the United States. Using this international experience to guide the selection of variables and model forms, the research team performed a considerable amount of safety model development in this research:

- **Development of intersection-level safety performance functions (SPFs) that can be applied at a planning level for estimating the incidence of crashes.** This development involved testing existing models and (1) recalibrating them if feasible or (2) developing new models if the existing models were determined to be an inadequate fit to U.S. data. This latter step capitalized on insights gained from previous modeling experiences to identify model deficiencies resulting from omitted variables, incorrect functional forms, overfit models, and lack of causal variables.
- **Development and evaluation of approach-level SPFs to explore design relationships and assessment of the**

value of approach-level predictive models. This development involved the same sequence of steps used for the intersection-level model development.

- **Exploration of the potential for speed-based safety models.** The concept of a speed-based model that relates safety performance to absolute speeds and/or relative speeds (speed consistency) was pursued with the hope of providing an intermediate link to both safety and operational performance. The rationale is that speed profiles are a manifestation of the driver's response to a design. The work included testing and calibrating models for linking crashes to the speed profile and the speed profile to roundabout characteristics.
- **Development of an expanded comparison of intersection safety before and after installation of roundabouts.** This comparison between roundabouts and the form of control that preceded their installation was disaggregated as much as possible (e.g., urban versus rural, one lanes versus two lanes) to improve its utility to practitioners.

Motorized Design Criteria

The approach to developing updated design criteria had two major components: speed estimation, and incorporation of safety and operational findings into design criteria. Both efforts were conducted under the premise that the overall design methods currently in use in the United States are sound and that any findings from this study would supplement and augment those procedures, not completely replace them (unless findings suggested otherwise).

The approach used for speed estimation in roundabouts involved collecting and comparing spot speed data in the field for various movements through the roundabout at key points along their paths. These speeds were then compared to current prediction techniques presented in FHWA's *Roundabouts: An Informational Guide* (*1*) to test the overall veracity of the current methods and to propose alterations as needed to improve the fit of the models to the field data.

For overall assessments of the effects of safety and operational findings on design, three approaches were used. First, an overall set of descriptive statistics for the sites in the study were prepared to assess the overall safety performance of each roundabout by its general configuration (e.g., single-lane versus multilane). Second, the safety and operational prediction models developed in their respective modeling efforts were examined for the relevance of various geometric parameters useful in design (e.g., entry width). Third, anecdotal evidence was used where modeling efforts were not sufficient to provide insight on potential relationships between the design of the roundabout and its potential safety and/or operational performance.

Pedestrian and Bicyclist Analysis Approach

The approach to this study was to collect data related to pedestrian, bicyclist, and motorist behaviors from enough locations and for enough pedestrians and bicyclists to answer the questions posed in the introduction of this report. The analysis produced a series of descriptive statistics from the acquired and derived data for each site, which defined the actions and behaviors of pedestrians, bicyclists, and motorists. These behaviors were then compared across sites to determine which locations should be reviewed more closely. Those sites that appeared to produce behaviors substantially different from the mean values for like sites were reviewed to determine if there were geometric or operational features at those locations that may have contributed to the observed behaviors.

In addition to comparing the roundabout sites, the research team also compared the results from the pedestrian analysis in this study to those of a study being conducted for FHWA, titled "Safety Index for Assessing Pedestrian and Bicyclist Safety at Intersections" (*3*). Specifically, the pedestrian time and behavior results from the roundabout approaches in this study were compared to similar data that were collected for two-way-stop–controlled, all-way-stop–controlled, and signalized intersections within the FHWA research study. The goal of this supplemental analysis was to shed light on any differences or similarities among these types of intersections with respect to pedestrian behaviors.

The objectives of the observational analysis were to characterize how pedestrians and bicyclists interact with motor vehicles at roundabouts, assess safety on the basis of these observations, and determine if there is an association between the observed behaviors and the geometric and/or operational characteristics. The following specific questions were addressed for pedestrians:

- How do pedestrians behave when crossing the leg of a roundabout? How do they respond to vehicles when preparing to cross or crossing the street? Do they cross within the crosswalk? Do they cross in one stage or two stages (using the splitter island as a refuge area)?
- What is the yielding behavior of motorists when they encounter a pedestrian who is crossing or waiting to cross?
- Did the behaviors of motorists and pedestrians create unsafe situations? Are there conflicts between motorists and pedestrians, and what are the underlying causes?
- How do the behaviors of pedestrians and motorists at roundabouts, which are yield controlled, compare to the behaviors of pedestrians and motorists at other types of crossings, including those with no control, stop control, or signal control?

- What are the geometric or operational characteristics that tend to cause problems for pedestrians or that tend to result in safer and more accessible designs? Are there differences in behaviors between the entry side and exit side of a leg? Are there differences in behaviors between one-lane and two-lane legs?
- Do any of the characteristics differ by region of the country?

The questions are similar for bicyclists crossing a leg. However, for bicyclists entering the roundabout, additional questions were addressed:

- How do motorists and bicyclists interact on the approach to the roundabout and within the circulating lane? Where do bicyclists position themselves; does the bicyclist "take the lane"?
- Are there conflicts or avoidance maneuvers on the approach or within the circulating lane?
- What types of behaviors do bicyclists exhibit that raise safety concerns (e.g., wrong-way riding, incorrect left turns)?

Marketing Materials

In addition to funding the scope of work, the AASHTO Standing Committee on Research approved additional funding to develop marketing materials for roundabouts. These materials consist of a series of self-guided Microsoft® PowerPoint™ presentations that can be used as is or adapted as needed to assist in communicating roundabout concepts to political and technical decision-makers. These presentations are available from the TRB website (http://www.trb.org/news/blurb_detail.asp?id=7086).

Literature Review

To support the research approach for this project, this report presents an extensive literature review that was conducted to support the two major efforts to model safety and operational performance. In addition, this report includes a brief summary of current design guidance in use in the United States to provide background for the design recommendations. Additional supplemental literature for other components of this study is referenced in their respective discussions of findings.

Safety Prediction Models

To date, most of the research and literature dealing with the safety of roundabouts has focussed on the relative change in safety following the conversion of conventional stop- and/or signal-controlled intersections to roundabouts. The explicit quantification of roundabout safety, measured in terms of expected crash frequency, has thus far not received equal attention. However, for the designer, understanding the relationships between roundabout design features and crash frequency is imperative.

This report reviews, by country of origin, published models that address the relationships between roundabout geometry and other factors, and safety. These models originate from the United Kingdom, Australia, France, and Sweden. Also reviewed are studies on the safety effect of converting conventional intersections to roundabouts.

A summary of the safety models included in the review is provided in Table 1. Appendix A contains a comprehensive review of each source, by country of origin, followed by a summary indicating how useful the insights from this review were in guiding the research effort. (All appendixes have been published as *NCHRP Web-Only Document 94* available on the TRB website [http://www.trb.org/news/blurb_detail.asp?id=7274]).

A summary of the effect of each parameter according to the models from the United Kingdom, Australia, and Sweden is provided in Table 2. The table is broken into common and unique measures for each model. The French model (SETRA), which is not tabulated here, related only one variable (average annual daily traffic [AADT], which had a positive effect) in a single model for all crash types combined. Measures common to two or more of the models are volume, pedestrian volumes, number of lanes on approach, number of circulating lanes, radius of the central island, radius of vehicle path, and approach curvature or deflection. Most factors, with the exception of the radius of the central island, were found to have similar effects on safety (i.e., same positive or negative direction).

The literature review provided insight on how many sites may be needed for direct calibration. For example, models for other intersection types were typically based on samples of 300 to 450 sites. Conversely, the UK model for four-leg roundabouts used only about 80 sites (*4*), but there was quite a large variation in some of the key variables. On this basis, the research team confirmed that the development of safety models for U.S. roundabouts would be a challenging task given the relatively few U.S. installations and the low numbers of crashes at them. Therefore, the research team concluded it would need to consider alternatives to the direct calibration of models relating crashes at roundabouts to all of the geometric and operational characteristics that may affect their safety.

For the direct calibration of models, it was evident from the review that, even if large sample sizes were available, the characteristics of interest would need to vary enough to allow the relationship between crashes and these variables to be modeled. These difficulties appeared to be magnified

Table 1. Summary of safety models.

Country and Author	Sample Size	Constant Features	Variable Features	Model	Input Parameters
United Kingdom: Maycock & Hall (*4*)	84	• Four legs • Single grade • Circular island • No unusual features	• Island size • Speed	Total crashes/ roundabout	• Vehicle AADT
				Total crashes/ crash type	• Vehicle AADT
				Total crashes/ leg/crash type (geometric)	• Vehicle AADT • Pedestrian volume • Entry width • Angle • Sight distance • Approach curve • Gradient • Radius • Percentage of motorcycles
Australia: Arndt (*5, 6*)	100	None	• Number of legs • Number of lanes • Urban/Rural • Island shape • Speed	Total crashes/ leg/crash type	• Vehicle AADT • Number of lanes • Speed variables • Vehicle path radius • Side friction
Sweden: Brüde & Larsson (*7*)	650	N/A	• Number of legs • Number of lanes • Speed limit	Crashes/million of entering vehicles	• Vehicle AADT
France: SETRA (*8*)	N/A	N/A	N/A	Total crashes/ roundabout	• Vehicle AADT

Legend:
AADT = Average annual daily traffic; N/A = Not available

in the modeling of different crash types. However, the various crash types needed to be modeled, as others have done, to guard against the opposite effects of a variable being masked. For example, increased entry deflection might reduce entering-circulating crashes but increase rear-end crashes (though to a lesser degree).

Most important, through the literature review, a wide array of variables for the safety analysis was identified. This list was useful in guiding the data collection and modeling efforts.

Review of Before-After Safety Studies

The research team also reviewed studies on the safety effect of converting conventional intersections to roundabouts and found that the results of these studies are usually reported without indicating whether regression-to-the-mean biases were considered in the analysis. Further, in most cases, the research team was unable to determine if this bias exists. Thus, the reader is cautioned to accept the results summarized here in the spirit in which this section is provided—to provide a flavor for the safety benefits of roundabouts. The decision to report these results in spite of possible reservations was based on a belief that, with the very large reductions that were consistently observed, the benefits of roundabouts would remain substantial if regression-to-the-mean effects were removed and any other methodological limitations were to be overcome. Details on studies of conversions from other forms of intersections can be found in Appendix A.

The one definitive study of U.S. conversions conducted for the Insurance Institute for Highway Safety (IIHS) (*9*), and subsequently updated for the New York State Department of Transportation (NYSDOT) (*10*), was based on a rather small sample size. As such, only limited disaggregate analysis could be done to try to isolate the geometric factors associated with the greatest safety benefits of roundabout construction. While some of these factors have been isolated in evaluations outside of the United States, that knowledge may not be directly transferable. In addition, several of those studies had methodological limitations. The review of the previous studies did provide useful insights for guiding the disaggregated before-after analysis for this study. Useful lessons were learned from the pitfalls and limitations of many of those studies (e.g., small sample sizes, ignoring regression to the mean, and improperly accounting for traffic volume changes over time). These lessons emphasized the need for, and the use to be made of, recent advances in safety estimation methodology aimed at overcoming these limitations.

Capacity Models

Capacity is a required input to delay and queuing models. In terms of existing U.S. capacity methodologies, the HCM

Table 2. Summary of geometric, traffic, and other characteristics affecting safety.

Measure	United Kingdom (Maycock & Hall)					Australia (Arndt)					Sweden (Brüde & Larsson)		
	SV	APP	Ent/C	Other	Ped	SV	RE	Ent/C	Ext/C	SS	All	Cyclist	Ped
Measures Common to All Models[1]													
AADT/volume	+	+	+	+	+	+	+	+	+	+	+		
Pedestrian volumes					+								+
Number of approaching lanes							+				+	+	+
Number of circulating lanes								+			+	+	+
Radius of central island			–					+			See Note 2	–	
Radius of vehicle path	–	–				–							
Approach curvature or deflection	–							–	–				
Measures Specific to the United Kingdom Models (Maycock & Hall) (4)													
Road width at entry	*/+	*/–	+/–										
Percentage of motorcycles			+	+									
Angle to next leg		*	–										
Gradient		+/+%	+/–%										
Sight distance	+	+											
Weaving length between splitter islands		*			*								
Distance to first sight of roundabout					*								
Average flare length		*											
Measures Specific to the Australia Models (Arndt) (5, 6)													
Length of vehicle path						+							
85^{th} percentile speeds						+	+	+	+				
Reduction in 85^{th} percentile speed						+							
Potential side friction										+			
Measures Specific to the Sweden Models (Brüde & Larsson) (7)													
Three legs instead of four legs											–		
Posted speed limit	*				*						+		
Presence of bicycle crossings												–	

Legend:
SV = single vehicle; *APP* = approaching; *Ent/C* = crashes between an entering and a circulating vehicle; *Other* = other non-pedestrian crashes; *Ped* = pedestrian crashes; *RE* = rear-end crashes on approach; *Ext/C* = crashes between an exiting vehicle and a circulating vehicle at multilane roundabouts; *SS* = sideswipe crashes on two-lane segments.
+ = an increase in this measure increases crash frequency
– = an increase in this measure decreases crash frequency
* = the measure had a significant relationship with crash frequency but the relationship was not specified

Notes:
[1]The French model (SETRA) (*8*) is inappropriate to tabulate here because it related only one variable (AADT) in a single model for all crash types combined. AADT had a positive effect.
[2]Optimum 10 m to 25 m

includes a gap acceptance model limited to single-lane roundabouts, and it does not provide any guidance on delay, queues, or level of service. The methods in FHWA's *Roundabouts: An Informational Guide* (*1*) for one- and two-lane roundabout capacities were derived using the UK empirical model with assumed values for the six geometric input parameters. The German empirical capacity relationship was recommended for the operational analysis of an urban compact roundabout. These models were intended to be provisional until further research could be performed on U.S. roundabouts.

A summary of the international capacity models is shown in Table 3. These models are either gap acceptance or linear/exponential empirical relationships. Except for the UK model, there are few geometric parameters.

Details on the types of capacity models in use, as well as a survey of international practices in estimating capacity, can be found in Appendix B.

Overall Literature Review Summary

The literature review provided the following useful insights that were used to guide the conduct of the NCHRP 3-65 research:

- A wide array of variables for the safety and operational analyses were identified. The list was useful in guiding the data collection and modeling efforts.
- While safety and operational prediction models have been developed successfully in other countries, it was unclear if

Table 3. Summary of operational models.

Country	Author	Type	Applicability	Input Parameters	Comments
Germany	Wu *(11)*	Gap Acceptance	One to three lanes	• Circulating flow • Number of lanes • Critical headway • Follow-up headway • Minimum gap	Recommended model in Germany. Based on Tanner (*12*)
	Brilon et al. (*13*)	Linear Regression	One to three lanes	• Circulating flow	Refined for one lane
	Brilon et al. (*14*)	Linear Regression	One to three lanes	• Circulating flow	No longer applicable for one lane
	Stuwe (*15*)	Exponential Regression	One to three lanes	• Circulating flow • Number of lanes • Number of legs • Diameter • Travel distance	Limited geometric range applicable
Switzerland	Simon (*16*)	Linear Regression	One lane, bus lane	• Circulating flow	Not applicable to two or more lanes
	Lausanne, as reported in Bovy et al. (*17*)	Linear Regression	One to three lanes	• Circulating flow • Entering flow • Conflict length	Three unique formulas; one lane limited to D = 22-32 m
USA	HCM (*2*)	Gap Acceptance	One lane	• Circulating flow • Critical headway • Follow-up headway	Provisional method. Based on Harders (*18*)
	Robinson et al. *(1)*	Linear Relationship	Urban Compact	• Circulating flow	See Brilon et al. (*13*)
		Linear Relationship	One lane, Diameter = 30-40 m	• Circulating flow	See Kimber (*19*)
		Linear Relationship	Two lanes, Diameter = 55-60 m	• Circulating flow	See Kimber (*19*)
UK	Kimber (*19*)	Linear Regression	All	• Circulating flow • Entry width • Approach half-width • Effective flare length • Entry angle • Entry radius • Diameter	Large sample of observed capacities
France	CETE Quest (*20*)	Exponential Regression	All	• Circulating flow • Exiting flow on leg • Entry width • Width of splitter • Width of circulatory lane	Girabase method. Most widely used in France
	Louah (*21*)	Linear Relationship	N/A	• Circulating flow • Exiting flow on leg	
	CETUR (*22*)	Linear Relationship	One lane	• Circulating flow	Adjustments have been developed for different geometric factors

Table 3. (Continued).

Country	Author	Type	Applicability	Input Parameters	Comments
Netherlands	CROW (*23*)	Range from macro to micro models	N/A	N/A	Approximate and calculation methods
	CROW (*23*), Botma (*24*)		One lane	• Circulating flow • Exiting flow on leg • Number of bicycles	
	Arem & Kneepkens (*25*)	Gap Acceptance		• Circulating flow • Exiting flow on leg • Critical headway • Follow-up headway • Minimum gap	Believed to be poorly researched. Based on Tanner (*12*)
Sweden	CAPCAL2 (*26*)	Gap Acceptance	One to two lanes	• Percentage of heavy vehicles • Critical headway • Follow-up headway • Minimum gap • Proportion of random arrivals • Length of weave area • Width of weave area	Guidebook based on Australian methods; Critical headway - based on geometry
Israel	Polus & Shmueli (*27*)	Exponential Regression	One lane	• Number of legs • Number of lanes • Speed limit	Units are not specified
Australia	Troutbeck (*28*)	Gap Acceptance	One to three lanes	• Circulating flow • Turning flow • Entry flow • Number of lanes • Entry width • Diameter • Critical headway • Follow-up headway	Separate equations for left and right lanes. Insufficient sites to develop linear regression equations
Austria	Fischer (*29*)	Linear Regression	One lane, Diameter = 23-40 m	• Circulating flow	Similar to Swiss method

these models were directly transferable to the United States. Therefore, a considerable amount of model testing and subsequent model development was required for this research. However, direct transfer of the models did appear feasible and the experience from elsewhere could be used, at least, to guide the selection of variables and model forms.

- New safety and operational models should be sensitive to the volume and variation of data acquired by past studies, recognizing that the U.S. database is inherently less rich at this stage in U.S. roundabout development.
- Few before-after safety studies of roundabout installations have been methodologically sound. Lessons were learned from the pitfalls and limitations of these studies (e.g., small sample sizes, ignoring regression to the mean, improperly accounting for traffic volume changes over time). Even the later before-after studies that learned some of these lessons suffered from small sample sizes that limited the disaggregate analysis aimed at identifying the factors associated with the safety benefits of roundabouts. However, it is feasible and useful to capitalize on the recent advances in safety estimation methodology and a now rich sample of U.S. conversions to do, as part of NCHRP 3-65, a before-after study that would in a disaggregate analysis identify a larger number of factors associated with the safety benefits of roundabouts than was possible before.

CHAPTER 2

Data Characteristics

This chapter describes the process used to establish an overall inventory of roundabouts in the United States, the selection of sites for data collection, and the various types of data collected at specific sites. This database serves three major purposes:

- It supports the development of United States-based safety and operational models completed as part of this project.
- It provides a foundation for additional research into topics beyond the scope and budget of this project.
- It establishes a baseline of U.S. roundabout performance during 2003, the year during which most field data were collected. Future research will be able to compare conditions at that time with those experienced in 2003 to determine trends in various measures over time.

The following sections discuss the development of this database, including the overall inventory and collection of the various geometric, operational, speed, and safety elements.

Site Inventory

One of the products of this research project is an updated site inventory that contains information on as many roundabouts as possible during its development. Table 4 provides a summary of the database, compiled by the project team, of the 310 known roundabouts that existed in the United States as of 2003; the locations of these roundabouts are shown graphically in Figure 1. This database was initially developed by a team led by Rensselaer Polytechnic Institute (RPI) for a project conducted for the NYSDOT and supplemented by information collected for this research project (*30*). Most (94%) of these roundabouts are located in urban or suburban areas, with more than half located in the western United States. The most common geometric configuration (more than two-thirds of the roundabouts) consists of a one-lane circulating roadway and four legs. Sixty-one percent of the roundabouts were converted from some form of stop control, while nearly a third were newly constructed intersections. Nearly all of these roundabouts were constructed during the past 10 years, with 46% opening between 2000 and 2003. A complete listing and description of the site inventory used for this project is given in Appendix C.

Table 5 lists the subset of known sites used for the analysis in this study. Each of these sites was assigned a unique site identification code consisting of a two-letter state abbreviation and a two-digit identification number (e.g., MD01). This code was used in combination with a cardinal direction designation for a given leg (e.g., MD01-N) and/or a video number (e.g., MD01-N2) to identify specific videos for a given leg.

Safety Data

Safety databases were required for three purposes:

- To develop intersection-level crash prediction models
- To develop approach-level crash prediction models
- To conduct a before-after study of roundabouts converted from signal or stop control

For a roundabout to be eligible for inclusion in the sample used for each of the three purposes outlined above, the data available had to meet minimum inclusion criteria that varied based on the model under consideration. For developing intersection-level prediction models, crash and traffic volume data and basic geometric information such as number of legs and number of lanes had to be available for a period of time after the roundabout was constructed. The same information was required for approach-level models; however, the data needed to be available at the approach level, and more detailed geometric data were required. For the before-after study, it was necessary to have, at a minimum, AADT volumes for either the before or after period, the construction dates,

Table 4. Characteristics of modern roundabouts located in the United States (2003).

Characteristics	Number	Percentage of total
Total number	310	
Setting		
• Urban	103	36%
• Suburban	164	58%
• Rural	16	6%
Number of legs		
• 6	4	1%
• 5	16	5%
• 4	197	68%
• 3	70	24%
• 2	4	1%
Number of circulating lanes		
• 3	5	2%
• 2	72	25%
• 1	213	73%
Previous intersection		
• One-way stop	30	19%
• Two-way stop	49	32%
• All-way stop	16	10%
• Signal	14	9%
• None	46	30%
Year created		
• 2000-2003	70	46%
• 1995-1999	70	46%
• 1994 or earlier	12	8%
Geographic location (zip code)		
• Northeast (0,1)	24	8%
• Mid-Atlantic (2)	45	15%
• South, Southeast (3,7)	32	10%
• Midwest (4,5,6)	39	13%
• Mountain West (8)	94	30%
• Pacific Coast (9)	76	25%

Note: Not all characteristics are available for all sites; this explains why the totals for each characteristic add up to less than 310, the total number of roundabouts in the database. For example, setting data are available for 283 of the 310 roundabouts. The percentages cited for urban, suburban, and rural settings add up to 100% of the sample of sites for which data for this characteristic is available. The number of legs and geographic location data do not add to 100% because of rounding.

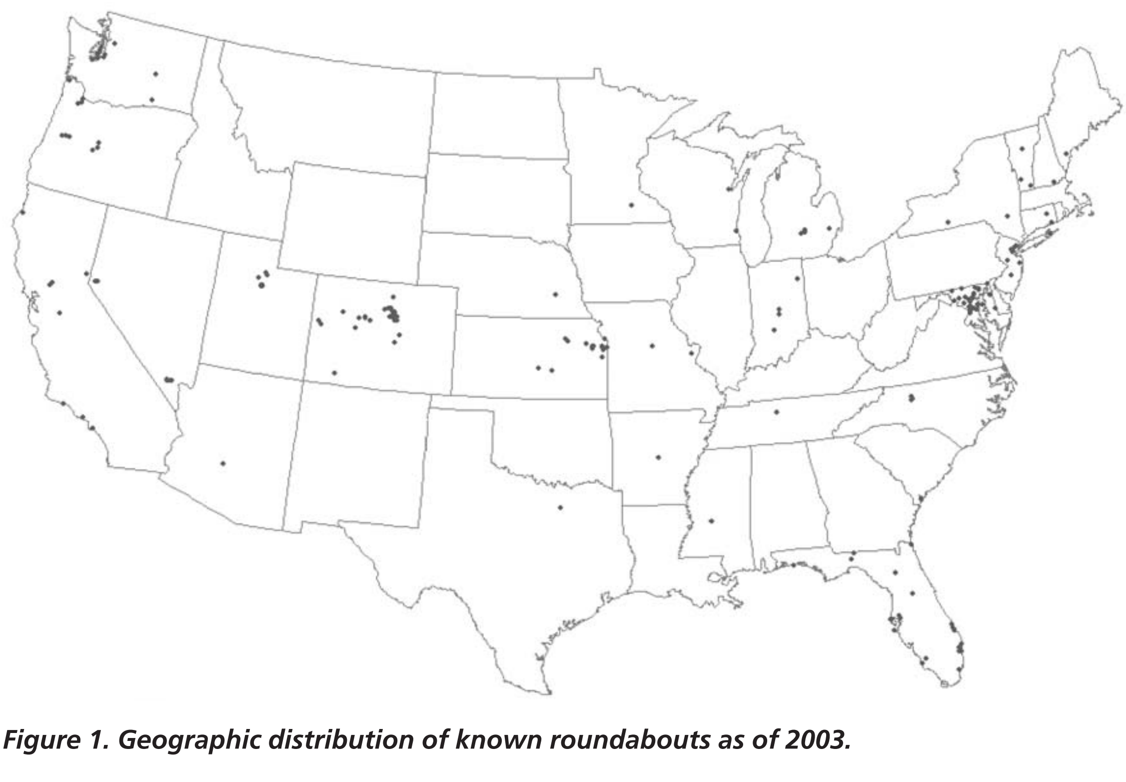

Figure 1. Geographic distribution of known roundabouts as of 2003.

Table 5. Subset of sites used for analysis.

Site ID	State	City	County	Intersection	Setting	Legs	Lanes	Safety	Operational	Speed	Pedestrian	Bicyclist
CA06	CA	Davis	Yolo	Anderson Rd/Alvarado Ave	U	4	1	X				X
CA10	CA	Long Beach	Los Angeles	Pac Coast Hwy/Hwy 19/Los Coyotes Diag.	U	4	3	X				
CA11	CA	Modesto	Stanislaus	La Loma/James St./G St.	U	5	1	X				
CA17	CA	Santa Barbara	Santa Barbara	Milpas St/US 101 NB Ramps/Carpinteria St	U	5	2					X
CA23	CA	Modesto	Stanislaus	W Rumble Rd/Carver Rd	U	4	1	X				
CO01	CO	Eagle	Eagle	SH-6/I-70 spur/Eby Creek Rd	R	4	1			X		
CO02	CO	Golden	Jefferson	South Golden Road/Johnson Rd/16th Street	U	4	2			X		
CO03	CO	Golden	Jefferson	South Golden Road/Utah St.	U	4	2			X		
CO04	CO	Aspen	Pitkin	SH 82/Maroon Crk/Castle Crk	S	4	2			X		
CO06	CO	Avon	Eagle	Avon Rd./Beaver Creek Blvd.	U	4	3	X				
CO07	CO	Avon	Eagle	Avon Rd./Benchmark Road	U	4	2	X				
CO08	CO	Avon	Eagle	Avon Rd./I-70 Eastbound Ramp	U	4	2	X				
CO09	CO	Avon	Eagle	Avon Rd./I-70 Westbound Ramp	U	4	2	X				
CO10	CO	Avon	Eagle	Avon Rd./U.S. Hwy 6	U	4	2	X				
CO49	CO	Vail	Eagle	Chamonix Rd/I-70 EB Ramps/S Frontage Rd	S	6	2	X				
CO50	CO	Vail	Eagle	Chamonix Rd/I-70 WB Ramps/N Frntge Rd	S	5	2	X				
CO51	CO	Vail	Eagle	Vail Rd/I-70 EB Ramps/South Frontage Rd	S	5	2	X	X			
CO52	CO	Vail	Eagle	Vail Rd/I-70 WB Ramps/N Frntg/Sprddle Cr.	S	6	3	X				
CT01	CT	Killingworth	Middlesex	Rte 80/Rte 81	R	4	1	X				
CT04	CT	N. Stonington	New London	Rte 2/Rte 184	U	4	1	X				
FL01	FL	Amelia Island	Nassau	SR AIA/Amelia Island Plantation	S	4	1					
FL02	FL	Boca Raton	Palm Beach	Cain Blvd/Boca Raton Dr	S	4	1	X				
FL09	FL	Bradntn Bch	Manatee	SR 789/Bridge St	S	3	1	X				
FL11	FL	Clearwtr Bch	Pinellas	SR 60/Coronado/Mandalay/Poinsettia	U	5	2		X		X	X
FL14	FL	Ft Wltn Bch	Okaloosa	Hollywood Blvd/Doolittle Blvd	U	3	1	X				
FL15	FL	Gainesville	Alachua	SE 7th Street/SE 4th Avenue	U	4	1	X				
KS01	KS	Olathe	Johnson	Sheridan St./Rogers Rd	U	4	2	X				
KS02	KS	Hutchinson	Reno	23rd Ave./Severence St.	U	4	1	X				
KS05	KS	Lawrence	Douglas	Monterey Way/Harvard Rd	S	3	1	X				
KS09	KS	Manhattan	Riley	Candlewood Dr/Gary Avenue	S	4	1	X				
KS10	KS	Manhattan	Riley	Kimball Ave/Grand Mere Parkway	S	3	1	X				
KS15	KS	Overland Park	Johnson	110th St./Lamar Ave.	S	4	2	X				
KS16	KS	Paola	Miami	K-68/Old Kansas City Rd/Hedge Lane	R	5	1	X				
MD01	MD	Bel Air	Harford	Tollgate Rd./Marketplace Dr.	S	3	1	X		X		
MD02	MD	Leeds	Cecil	MD 213/Leeds Rd/Elk Mills Rd (Lanzi Cir.)	R	4	1	X		X		
MD03	MD	Jarrettsville	Harford	MD 24/MD 165	R	4	1	X		X		
MD04	MD	(unincorporated)	Baltimore	MD 139 (Charles St.)/Bellona Ave	U	4	2	X	X	X		
MD05	MD	Towson	Baltimore	MD 45/MD 146/Joppa Rd	U	5	2	X	X		X	
MD06	MD	Lothian	Anne Arundel	MD 2/MD 408/MD 422	R	4	1	X	X	X		
MD07	MD	Taneytown	Carroll	MD 140/MD 832/Antrim Blvd	S	4	1	X	X	X		
MD08	MD	Annapolis	Anne Arundel	MD 450/Spa Rd./Taylor Ave	U	4	2	X				
MD11	MD	(unincorporated)	Baltimore	MD 372/Hilltop Circle (UMBC)	U	4	1	X				
MD12	MD	Bel Air	Harford	MD 7/Holly Oaks Drive	S	3	1	X				
MD13	MD	Brunswick	Frederick	MD 17/A St/B St/Maryland Ave	U	5	1	X				
MD14	MD	Cearfoss	Washington	MD 63/MD 58/Cearfoss Pike	R	4	1	X				
MD15	MD	Ellicott City	Howard	MD 100 EB Ramps/MD 103	S	4	1	X				
MD16	MD	Ellicott City	Howard	MD 100 WB Ramps/MD 103	S	4	1	X				
MD17	MD	Ellicott City	Howard	MD 100 WB Ramps/MD 104	S	4	2	X				
MD18	MD	Ellicott City	Howard	MD 100 WB Ramps/Snowden River Pkwy	S	4	1	X				
MD19	MD	Federalsburg	Caroline	MD 307/MD 313/MD 318	R	4	1	X				
MD25	MD	Lisbon	Howard	MD 94/MD144	R	4	1	X				
MD26	MD	Lisbon	Howard	MD 94/Old Frederick Rd	R	4	1	X				

Table 5. (Continued).

Site ID	State	City	County	Intersection	Setting	Legs	Lanes	Safety	Operational	Speed	Pedestrian	Bicycle
MD27	MD	Millington	Kent	US 301 NB Ramps/MD 291	R	4	1	X				
MD28	MD	Millington	Kent	US 301 SB Ramps/MD 291	R	4	1	X				
MD31	MD	Oak Grove	Pr. Georges	MD 193/Oak Grove Rd	U	3	1	X				
MD33	MD	Rosemont	Frederick	MD 17/MD 180	R	4	1	X				
MD38	MD	Stevensville	Queen Annes	MD 18/Castle Marina Rd	S	4	1	X				
MD39	MD	Temple Hills	Pr. Georges	MD 637/Good Hope Ave.	S	4	1	X				
MD40	MD	Temple Hills	Pr. Georges	MD 637/Oxon Run Dr.	S	4	1	X				
ME01	ME	Gorham	Cumberland	US 202/State Route 237	U	4	1	X	X	X		
MI01	MI	Okemos	Ingham	Hamilton Rd/Marsh Rd	S	3	2	X	X	X		
MI03	MI	East Lansing	Ingham	Bogue Street/Shaw Lane	U	4	2	X				
MO01	MO	Columbia	Boone	Business Loop/I-70	S	5	1	X				
MS01	MS	Jackson	Rankin	MS 475/Airport Rd/Old Brandon Rd	S	4	1	X				
NV01	NV	Las Vegas	Clark	Hills Cen. Dr./Vllg. Cen. Cir./Mdw. Hills Dr.	S	4	2	X				
NV02	NV	Las Vegas	Clark	Town Cen. Dr/Hualapai Way/Far Hills Ave.	S	4	3	X		X		
NV03	NV	Las Vegas	Clark	Town Cen. Dr./Village Cen. Cir./Lib. Hills Dr.	S	4	2	X			X	
NV04	NV	Las Vegas	Clark	Town Cen./Cyn. Run Dr/Banburry Cross Dr	S	4	3	X				
NV05	NV	Carson City	Carson City	5th St/Edmonds	R	4	1	X				
NV09	NV	Las Vegas	Clark	Carey Ave/Hamilton St	U	4	2	X				
NV10	NV	Las Vegas	Clark	Carey Ave/Revere St	U	4	2	X				
NV16	NV	Las Vegas	Clark	Lake South/Crystal Water Way	S	4	1	X				
NV18	NV	Las Vegas	Clark	Hills Drive/Longspur	S	3	2	X				
OR01	OR	Bend	Deschutes	Colorado Ave/Simpson Dr	U	4	1	X	X	X		X
OR04	OR	Bend	Deschutes	Century Dr/Colorado Ave/Chandler Ave	U	4	1	X				
OR07	OR	Bend	Deschutes	Mt. Washington Dr/Shevlin Park Rd.	S	4	1	X				
OR09	OR	Bend	Deschutes	Century Dr./14th St./Simpson Ave.	U	4	1	X				
OR15	OR	Eugene	Lane	Barger Dr/Green Hill Rd	S	3	1	X				
SC01	SC	Hilton Head	Beaufort	Whooping Crane Way/Main St	S	4	1	X				
UT02	UT	Orem	Utah	2000 South/Sandhill Rd	U	4	2				X	X
VT01	VT	Manchester	Bennington	Rte 7A/Equinox(Grand Union)	S	4	1			X	X	
VT02	VT	Montpelier	Washington	Main St./Spring St (Keck Circle)	U	3	1	X		X		X
VT03	VT	Brattleboro	Windham	RT 9/RT 5	S	4	2	X	X	X		
WA01	WA	Gig Harbor	Pierce	SR 16 SB Ramp/Borgen Blvd.	S	4	1		X	X		
WA02	WA	Gig Harbor	Pierce	Borgen Blvd/51st	S	4	1			X	X	
WA03	WA	Bainbridge Is.	Kitsap	High School Rd/Madison Ave.	U	4	1	X	X	X	X	X
WA04	WA	Port Orchard	Kitsap	Mile Hill Dr. (Hwy 166)/Bethel Ave	S	3	1	X	X	X		
WA05	WA	Sammamish	King	NE Inglewood Hill/216th Ave NE	S	4	1		X	X		
WA06	WA	Monroe	Snohomish	SR 522 EB Ramps/W. Main St./Tester Rd	S	5	2	X				
WA07	WA	Lacey	Thurston	I-5 NB Ramp/Quinault Dr/Galaxy Dr	S	4	1	X	X	X		
WA08	WA	Kennewick	Benton	27th Ave/Union St/Union Loop Rd	U	4	1	X	X	X		
WA09	WA	Gig Harbor	Pierce	SR 16 NB Ramps/Burnham Dr./Borgen Blvd.	U	6	2		X	X		
WA10	WA	Federal Way	King	Weyerhauser Way/33rd Pl./32nd Dr. S.	S	3	2	X				
WA15	WA	Lacey	Thurston	Marvin Rd/Britton Pkwy./Willamette Drive	S	4	2	X				
WA16	WA	Lacey	Thurston	College St. SE/45th Ave. SE	S	4	2	X				
WA17	WA	Lacey	Thurston	Marvin Rd./Hawk Prairie Rd.	S	4	1	X				
WA22	WA	University Pl.	Pierce	Grandview Dr/56th St W	S	3	1	X				
WA23	WA	University Pl.	Pierce	Grandview Dr/62nd Court W/Park Entrance	S	4	1	X				
WA24	WA	University Pl.	Pierce	Grandview Dr/Bristonwood Dr/48th St W	S	4	1	X				
WA25	WA	University Pl.	Pierce	Grandview Dr/Cirque Dr	S	3	1	X				
WA26	WA	University Pl.	Pierce	Grandview Dr/Olympic Blvd	S	4	1	X				
WA27	WA	University Pl.	Pierce	56th Ave./Alameda Ave. W/Cirque Dr.	S	4	1	X				
WI01	WI	Howard	Brown	Lineville Rd (CTH M)/Cardinal Ln	S	4	1	X				

Legend: U = urban, S = suburban, R = rural. Settings are approximate.
The complete 2003 site inventory can be found in Appendix C.

the control type before construction, and crash data for both the before and after periods.

Crash data at these roundabouts were gathered by three primary means:

- Crash records were gathered from local jurisdictions in the vicinity of all field data collection sites.
- Additional data were gathered via phone calls, e-mail, and traditional mailings to jurisdictions that might have roundabouts with significant crash histories (i.e., roundabouts that had been in operation for more than 1 year).
- Data were extracted from files created by RPI for the NYSDOT project.

The 90 roundabouts in the intersection-level crash dataset were selected based on the availability of crash data (either summaries or detailed crash records), basic geometric information (e.g., number of lanes, number of legs, and diameter), and total entering daily traffic volumes; all components were needed for the site to be included in the dataset. The majority of these 90 roundabouts were single-lane roundabout sites, in urban or suburban environments. In addition, the roundabouts studied have an average AADT of approximately 16,700 entering vehicles/day (low of 2,668 and a high of 58,800). Figure 2 characterizes the sites used in the intersection-level crash dataset. Figure 3 provides a summary of the frequency of crashes at sites in the intersection-level crash dataset. As shown, the majority of the roundabout crashes per year are occurring at urban, multilane roundabouts. Within the dataset, there was little difference in the frequency of crashes per year at the single-lane urban, suburban, and rural roundabouts.

Tables 6 and 7 characterize the approach-level model dataset, which is a subset of the total intersection-level dataset. A total of 139 legs were included in the approach-level dataset. These 139 legs were selected independently from the 90 roundabouts used for the intersection-level dataset

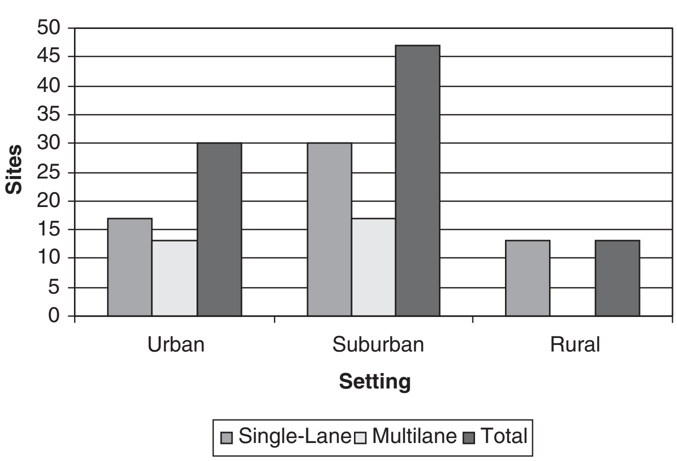

Figure 2. Summary of roundabout characteristics used for intersection-level safety analysis.

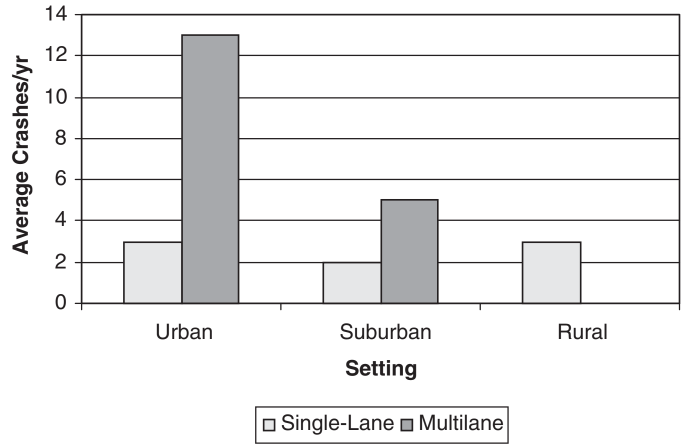

Figure 3. Intersection-level roundabout dataset—average crashes per year.

based on different data requirements. As noted previously, for a leg to be included in the dataset, all of the following data were needed: detailed crash records (e.g., police reports), detailed geometry (e.g., entry width, entry angle, approach curvature, etc.), and approaching and circulating daily traffic volumes. Table 8 provides a summary of the geometric data used in the approach-level modeling. The geometric data were developed from a manual review and reduction of data from as-built drawings of the roundabouts. Within the approach-level dataset, Figure 4 provides a summary of the types of crashes occurring. As shown, the majority of the crashes in the dataset occurred at multilane roundabouts and were either exiting-circulating or rear-end crashes.

Fifty-five roundabouts were used for the before-after study, as these were the roundabouts where both before- and after-conversion crash records were available. The two sources for the before-after dataset were the previously conducted before-after study for the IIHS (5) and new data collected for this project. A breakdown of this dataset by jurisdiction, control type before conversion, setting, and number of circulating lanes is shown in Table 9 and Figure 5. At these sites, before roundabout installation, there were 1,159 crashes; after installation, there were 726 crashes. The average length of time of the crash history before roundabout installation was 3.7 years; the average length of time of the crash history after installation of the roundabout was 3.3 years.

Operational Data

The overall inventory of roundabouts provided a rich source from which potential sites for the field data collection could be identified. The following criteria were used to identify these sites:

- An expectation of queuing on one or more of the roundabout approaches, representing capacity conditions

Table 6. Incidence of approach-level crashes by type.

Crash Type	Number of legs = 139 (at 39 roundabouts) Mean length of crash history = 3.8 years					
	All		Single Lane		Multilane	
	Incidence	Percentage	Incidence	Percentage	Incidence	Percentage
Entering-Circulating	141	23%	40	29%	101	22%
Exit-Circulating	187	31%	10	7%	177	38%
Rear-End on Approach	187	31%	42	30%	145	31%
Loss of Control on Approach	77	13%	42	30%	35	7%
Pedestrian	5	1%	1	1%	4	1%
Bicyclist	8	1%	3	2%	5	1%
Sum*	**605**	**100%**	**138**	**99%**	**467**	**100%**

*Percentages may not add to 100 because of rounding.

Table 7. Annual frequency of approach-level crashes by type.

Crash Type	Number of legs = 139 (at 39 roundabouts) Mean length of crash history = 3.8 years					
	All		Single Lane		Multilane	
	Maximum Crashes Per Year	Mean Crashes Per Year	Maximum Crashes Per Year	Mean Crashes Per Year	Maximum Crashes Per Year	Mean Crashes Per Year
Entering-Circulating	3.03	0.32	3.03	0.22	2.67	0.41
Exit-Circulating	9.09	0.57	9.09	0.57	7.67	0.97
Rear-End on Approach	5.00	0.40	2.00	0.17	5.00	0.64
Loss of Control on Approach	3.03	0.15	3.03	0.18	1.25	0.11
Pedestrian	1.00	0.01	0.14	0.02	1.00	0.03
Bicyclist	3.03	0.05	3.03	0.05	2.00	0.04

Table 8. Summary of approach-level geometric data used for safety analysis.

Variable	Minimum	Maximum	Mean	No. Legs
Inscribed Circle Diameter (ft)	85	300	144.1	139
Entry Width (ft)	12	49	22.2	138
Approach Half-Width (ft)	10	49	20.2	130
Effective Flare Length (ft)	0	308	27.5	134
Entry Radius (ft)	26	282	77.8	131
Entry Angle	0	45	19.2	129
Circulating Width (ft)	12	45	26.1	138
Exit Width (ft)	12	51	23.0	128
Departure Width (ft)	10	50	19.3	123
Exit Radius (ft)	21	285	82.0	115
Central Island Diameter (ft)	20	214	77.7	134
Angle to Next Leg	27	180	89.3	135
1/Entry Path Radius (1/ft)	-0.0100	0.0200	0.0058	123
1/Circulating Path Radius (1/ft)	-0.0300	0.0091	-0.0101	122
1/Exit Path Radius (1/ft)	0.0000	0.0252	0.0053	123
1/Left-Turn Path Radius (1/ft)	-0.0400	0.0244	-0.0184	120
1/Right-Turn Path Radius (1/ft)	0.0000	0.0364	0.0102	121
AADT	220	19,593	4,637	139

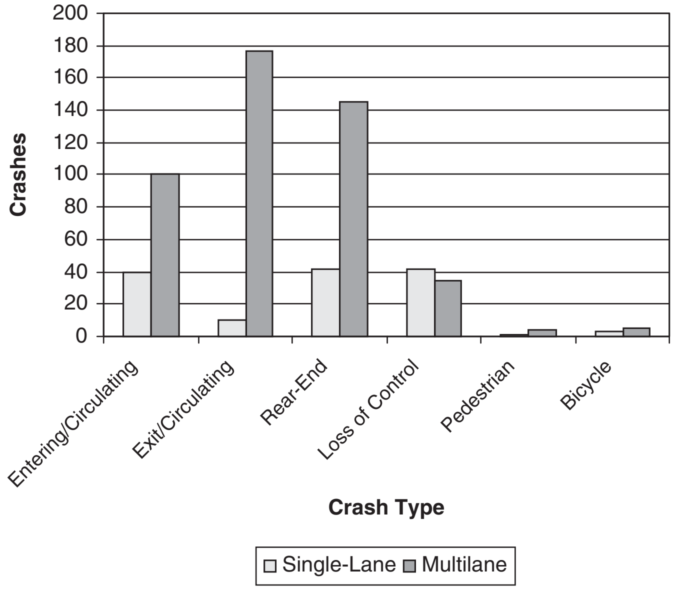

Figure 4. Approach-level crashes by type.

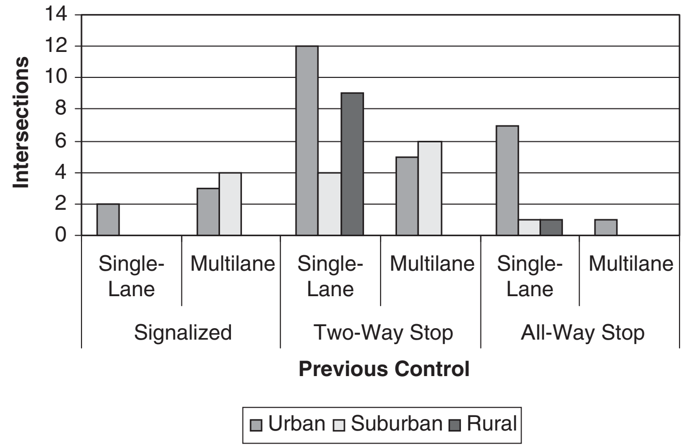

Figure 5. Dataset for before-after study by control type before conversion, setting, and number of circulating lanes.

- A balance between single-lane and multilane sites so that operational characteristics of both kinds of sites could be studied
- A range of other geometric conditions so that the effect of these conditions on operations could be studied
- A clustering of sites so that driving time to the sites could be minimized, thus maximizing the number of sites that could be studied

Table 10 shows a list of the 31 sites at which field video recordings were made during spring and summer 2003. Included in the table are the date of the site visit, the site ID, the intersection name, and the city and state in which the roundabout is located. A total of 34 hours of traffic operations was extracted including the entry flow, conflicting flow, exiting flow, accepted and rejected gaps, turning movement proportions, travel time for different movements, vehicle types, and delay.

A video recording system was designed to allow the team to record the movement of vehicles at the roundabouts selected for the operations study. The recording system included the following components:

- One omni-directional camera that provided a 360-degree view of the roundabout

Table 9. Dataset for before-after study by state, control type before conversion, setting, and number of circulating lanes.

State (Total Sites)	Signalized before						TWSC before						AWSC before					
	U		S		R		U		S		R		U		S		R	
	1L	2L	1L	2L	1L	2L	1L	2L	1L	2L	1L	2L	1L	2L	1L	2L	1L	2L
Colorado (9)				3						6								
Florida (4)	1						2						1					
Kansas (4)							1	1	1		1							
Maryland (17)		2					4	1	2		8							
Maine (1)							1											
Michigan (1)		1																
Mississippi (1)													1					
Missouri (1)													1					
Nevada (3)								1						1			1	
Oregon (4)							1						3					
S. Carolina (1)	1																	
Vermont (2)				1			1											
Washington (6)							1	2	1				1		1			
Wisconsin (1)							1											
TOTALS	2	3	0	4	0	0	12	5	4	6	9	0	7	1	1	0	1	0
	5		4		0		17		10		9		8		1		1	
	9						36						10					

Legend: TWSC = two-way-stop–controlled; AWSC = all-way-stop–controlled; U = urban; S = suburban; R = rural; 1L = one-lane; 2L = two-lane

Table 10. List of field sites for operational and speed data collection.

Date		Site ID	Intersection name	City and State
June	9	NV01	Hills Center Dr./Village Center Cir./Meadow Hills Dr.	Las Vegas, NV
	10	NV02	Town Center Dr./Hualapai Way/Far Hills Ave.	Las Vegas, NV
	11	NV03	Town Center Dr./Village Center Cir./Library Hills Dr.	Las Vegas, NV
	12	NV04	Town Center Dr./Banburry Cross Dr.	Las Vegas, NV
	17	CO01	SH-6/I-70 spur	Eagle, CO
	19	CO02	South Golden Road/Johnson Rd/16th St.	Golden, CO
	20	CO03	South Golden Road/Utah St.	Golden, CO
	30	MD01	Tollgate Rd. /Marketplace Dr.	Bel Air, MD
July	1	MD02	MD213 at Leeds Rd./Elk Mills Rd. (Lanzi Circle)	Leeds, MD
	2	MD03	MD24 at MD 165 (North Harford)	Jarrettsville, MD
	7	MD04	MD139 (Charles St.) at Bellona Ave.	Baltimore Co., MD
	8	MD05	MD45 at MD146/Joppa Rd.	Towson, MD
	9	MD06	MD 2 at MD 408/MD 422	Lothian, MD
	10	MD07	MD 140/MD 832/Antrim Blvd.	Taneytown, MD
	14	VT01	Route 7A/Equinox (Grand Union)	Manchester, VT
	15	VT02	Main St and Spring St. (Keck Circle)	Montpelier, VT
	16	VT03	Route 9/Route 5	Brattleboro, VT
	18	ME01	US 202/State Route 237	Gorham, ME
	23	MI01	Hamilton Rd/Marsh Rd.	Okemos, MI
	25	KS01	Sheridan St./Rogers Rd.	Olathe, KS
	28	CO04	SH 82/ Maroon Creek, Castle Creek	Aspen, CO
	30	UT01	1200 South/400 West	Orem, UT
	31	UT02	1200 South/Sandhill	Orem, UT
August	4	WA01	SR 16 SB Ramp Terminal (near Pioneer at Stinson)	Gig Harbor, WA
	5	WA02	Borgen Blvd/51st	Gig Harbor, WA
	6	WA03	High School Rd/Madison Ave.	Bainbridge Isl., WA
	7	WA04	Mile Hill Dr. (Hwy 166)/Bethel Ave.	Port Orchard, WA
	11	WA05	NE Inglewood Hill/216th Ave. NE	Sammamish, WA
	12	WA06	SR 522 EB Ramps/W. Main St./Tester Rd.	Monroe, WA
	13	WA07	I-5 off-ramp/Quinault Dr/Galaxy Dr.	Lacey, WA
	15	OR01	Colorado/Simpson	Bend, OR

Notes:
1. Other sites were included in the original field list. Bad weather prevented video recording at these sites.
2. The site ID includes the state in which the roundabout is located and the number of that site within a state.

- Three digital video cameras that focused on individual legs at the roundabout
- Two masts, each extendable to 30 ft, to which the video cameras were attached
- Four DVD-R recorders to record the video directly from the digital and omni-directional cameras at the site

Figure 6 shows an omni-directional camera (on the left) and a digital camera (on the right) mounted on the top of the mast. At the beginning of the field data collection, both masts were used, with two cameras located on the top of each mast. However, one mast and two digital cameras were destroyed during an unexpected windstorm in Colorado in late June 2003. Modifications were made to the remaining mast so that one omni-directional camera and three digital cameras could be mounted on that one mast.

Figure 7 shows a typical view of one leg taken by one of the digital cameras. This view shows both circulating vehicles and vehicles queued on the approach. All vehicle movements associated with this leg are clearly visible.

Figure 8 shows a typical view from the omni-directional camera. Vehicles on all four legs are shown, as well as vehicles circulating on the roundabout. This omni-directional view provides an excellent record of all vehicle movements, as well as of the intersection geometry and markings.

Using this video recording system, a total of 262 DVDs were recorded at the 31 sites: 166 DVDs of individual roundabout legs and 96 DVDs recorded of entire intersections using the omni-directional camera. The recordings made for the individual legs included 474 hours of traffic operations. Of the 166 legs recorded, 12 were located at three-lane sites, 58 were at two-lane sites, and 96 were at one-lane sites.

Geometric Data

To support the operational and safety model development, a wide range of geometric data were obtained for each site, as shown in Figure 9. Where possible, these data were collected using definitions consistent with those used for international safety and operational models. In addition, the type of pedestrian crosswalk, presence or absence of striping on the circulating roadway, lane configurations, and type of vertical geometry were noted.

Figure 6. Omni-directional camera and digital camera located at top of mast.

Figure 7. View of one leg from digital camera, Site WA03-S.

Figure 8. View from omni-directional camera, Site WA03.

Speed Data

Currently, speeds are predicted for a roundabout design by measuring speeds along the "fastest path," as defined in FHWA's *Roundabouts: An Informational Guide* (*1*). This path is assumed to be the fastest path traversable by a single *free-flow* vehicle without regard to pavement markings or other traffic. This methodology assumes no acceleration or deceleration between points of measure; as such, the resulting predicted speed represents a reasonable upper limit for the given radius, superelevation, and side friction factor. Appendix G provides details on the specific definitions for defining vehicle paths.

Sixteen single-lane and eleven multilane sites were used for this speed analysis. These sites were chosen to represent a range of geometry, surrounding land use, and volumes found at roundabouts, and the data items collected are summarized in the following paragraphs.

Spot speed data were collected for this project during the summer of 2003 at each location visited by the field data collection team. The speed data were collected using a radar gun, which recorded speeds of free-flow vehicles on each leg to the nearest 1 mph (1.6 km/h) at the following locations:

- At least 200 ft (60 m) upstream of the yield line
- At the yield line
- At the midpoint of the adjacent splitter island
- At the exit point of the roundabout

The number of actual observed data varied by location and leg, depending on the quantity of free-flow observations available and the time constraints of the field data collection team. For some legs, few or no data points were obtained; at other legs, the number of data points exceeded 30. Data points for entering, circulating, and exiting speeds were differentiated by turning movement (i.e., left, through, and right). All data were differentiated by vehicle type (i.e., passenger cars, trucks), and only passenger car data were used for this analysis.

Pedestrian and Bicyclist Data

The sites for the pedestrian and bicyclist observational study were selected from the large number of sites where digital video had originally been recorded as part of the overall field data collection effort. The sites chosen for the analysis were generally those with the greatest number of pedestrians and/or bicyclists. Data were also collected at several additional sites with known high volumes of pedestrians and/or bicyclists, which increased the number of observations and the range of geometric and operational conditions.

Ten specific legs, located at seven roundabouts, were chosen for the pedestrian study. Three of these legs were also used for the analysis of bicyclist movements. A total of 14 legs at seven roundabouts was selected for the study of bicyclists.

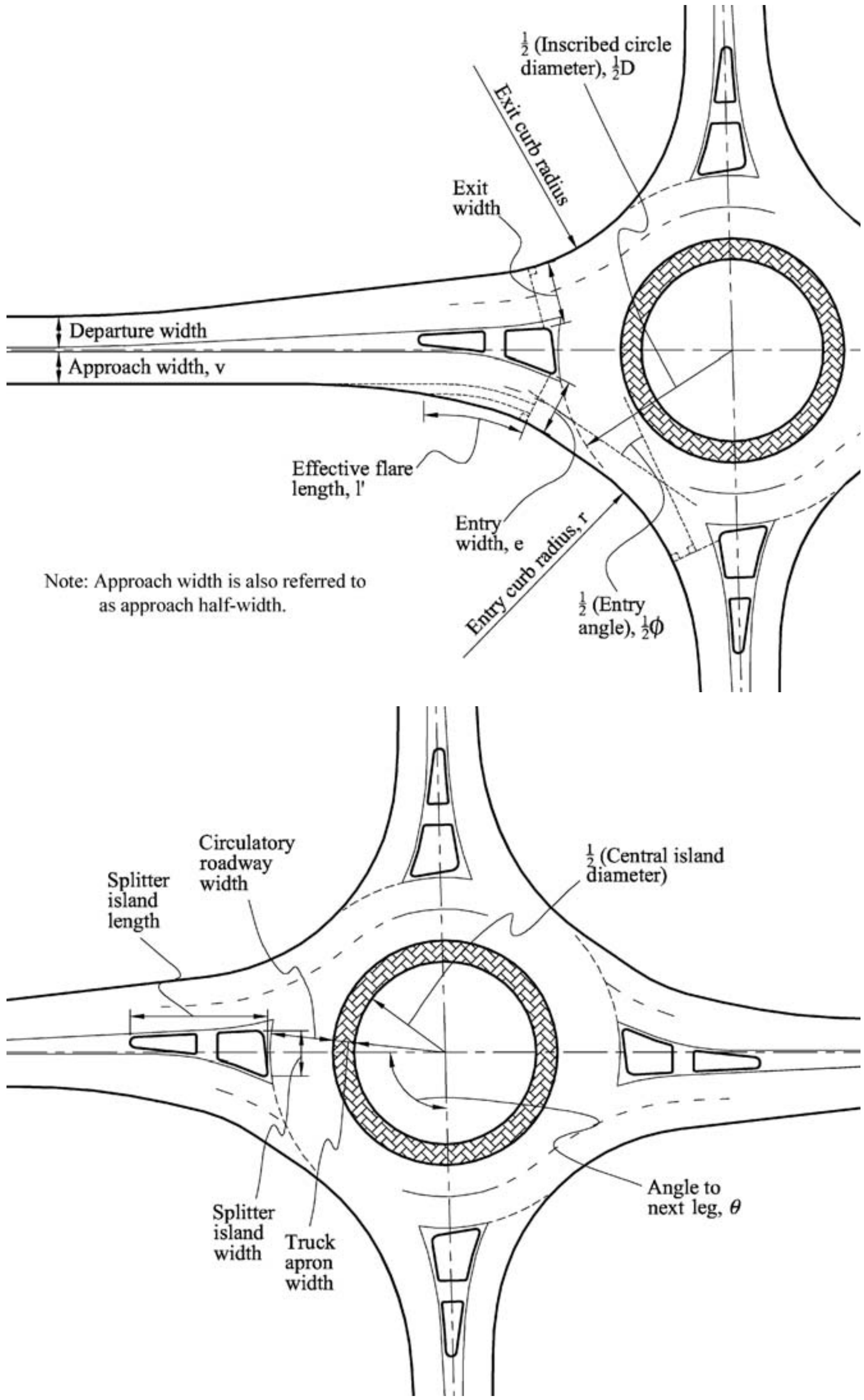

Figure 9. Geometric data obtained for each site.

A summary of all sites used for this analysis is provided in Table 11. A description and image of each leg can be found in Appendix C.

The observational data were acquired from DVDs and videotapes for the sites described previously. The data recorded for each event included the information necessary to attempt to answer the questions previously posed in the introduction. For each pedestrian crossing event, the following information was captured:

- Number of pedestrians crossing (to distinguish individuals from groups)
- Estimated age (adult versus youth)
- Start location (entry side versus exit side)
- Crossing type (within or outside the crosswalk)
- Arrival time (at the curb and prepared to cross)
- Start time (when the crossing was initiated)
- Wait time (difference in start time and arrival time)
- Splitter arrival/departure

Table 11. Summary of pedestrian and bicyclist observation sites.

Location	Intersection	Site/Video ID	Observation Period (min)	Ped. Events	Bike Events
Davis, CA	Anderson Rd/Alvarado Ave	CA06	99	—	89
Santa Barbara, CA	Milpas St/US 101 NB Ramps/Carpinteria St	CA17	120	—	57
Clearwater Beach, FL	SR 60/Gulf Blvd (SR 699)	FL11-E1	120	135	19
Towson, MD	MD 45/MD 146/Joppa Rd	MD05SW-S1	180	89	—
		MD05SW-W1	181	65	—
		MD05SW-NW1	180	38	—
Las Vegas, NV	Town Center Dr/Village Center Cir/Library Hills Dr	NV03-S1	220	22	—
Bend, OR	Colorado Ave/Simpson Dr	OR01-N1/N2	233/233	—	59
		OR01-S1	233	—	27
		OR01-W2	233	—	26
Orem, UT	2000 South/Sandhill Rd	UT02-E1	240	131	12
		UT02-W1	234	35	—
Manchester, VT	Main St (Rte 7A)/Grand Union	VT01-N1/N2	238/142	94	—
Brattleboro, VT	Rte 9/Rte 5	VT02-N1	240	—	39
		VT02-S1	240	—	58
		VT02-W1	240	—	49
Gig Harbor, WA	Borgen Blvd/51st Ave	WA02-E1	158	24	—
Bainbridge Island, WA	High School Rd/Madison Ave	WA03-N2	140	—	29
		WA03-S1/S2/S3	240/140/231	136	112
		WA03-E2/E3	141/231	—	84
		WA03-W1	231	—	30
Totals			**5,118**	**769**	**690**

Legend: Ped. = pedestrian

- Splitter time (difference in splitter departure and arrival times)
- End time (when the crossing was completed)
- Crossing time (difference in end time and start time)
- Rejected gaps (time between arrival time and the next vehicle reaching the crosswalk *and* time between each subsequent pair of vehicles until the pedestrian crosses)
- Accepted gap (time between arrival time and the next vehicle reaching the crosswalk after the pedestrian crosses *or* time between the last rejected gap vehicle and the next vehicle reaching the crosswalk after the pedestrian crosses)
- Pedestrian behaviors during crossing (none, hesitations, stops, retreats, runs)
- Motorist behaviors (yield-slow, yield-stop, does not yield)
- Conflicts (requiring either party to suddenly change course and/or speed)

For bicyclists, the data elements captured had to be expanded. While there were some bicyclists that crossed the leg like a pedestrian, many of the bicyclists captured were approaching the roundabout in the entry lane, departing the roundabout in the exit lane, or traversing the roundabout within the circulating lane. In addition to this basic event type, the following variables were captured for bicyclists:

- Position of bicyclist (within the lane on the leg)
- Motor vehicle presence (passing, trailing, leading, none)
- Entering bicyclist's behaviors (yield, did not yield, safe gap, unsafe gap)
- Exiting bicyclist's behaviors (lane/sidewalk position upon exit)
- Start location for crossing bicyclist (entry side versus exit side)
- Crossing type (within or outside the crosswalk)
- Conflicts (requiring either party to suddenly change course and/or speed)
- Other behaviors (e.g., wrong-way riding)

Appendix D includes more detail about each of the variables captured or derived for pedestrian and bicyclist observations.

Conclusion

The data described in this chapter are the basis for the safety, operational, and design analysis findings from this research. They also are intended to support future research efforts to further understanding of roundabouts in the United States. Discussion of additional research topics to use and expand upon the current database can be found in Chapter 7.

CHAPTER 3

Safety Findings

This chapter describes a number of safety modeling and evaluation tasks. First, the results of the investigation of the ability of non-U.S. prediction models to represent U.S. data are presented—separately for intersection-level and approach-level models. Next are presented the results of the efforts to develop intersection- and approach-level models for U.S. data collected for this project. The last two sections present the results for the modeling of safety as a function of speed and the results of the before-after study.

Ability of Existing Non-U.S. Models to Represent U.S. Data

To test the feasibility of international models, the models were calibrated and used to predict crashes at the U.S. roundabouts in the database. Statistical goodness-of-fit tests were then performed. Appendix E contains a summary of the goodness-of-fit measures and statistical assessments used in this analysis.

Ability of Non-U.S. Intersection-Level Accident Prediction Models to Represent U.S. Data

The general purpose of the intersection-level models is to predict the expected crash performance of an intersection during the planning level of analysis. These models are intended for comparing roundabouts with other forms of intersections and intersection control. As a result, the variables selected for use in the models are deliberately set to be the most basic variables, such as AADT, the number of legs, and the number of circulating lanes.

Existing models were tested with the U.S. data to assess their goodness of fit. The eight intersection-level models came from four sources: four models were based on data from Sweden, three from the United Kingdom, and one from France. These models are designated as follows:

- SWED-TOT1 and SWED-INJ1 (Equations A-10a and A-11a in Appendix A).
- SWED-INJ2 and SWED-INJ3 (Equations A-12a through A-12d in Appendix A).
- UK-INJ1, UK-INJ2, and UK-INJ3 (Equations A-1c through A-1f in Appendix A). The constant parameter for the semi-urban (30–40 mph, or 48–64 km/h) and rural (50–70 mph, or 80–112 km/h) models has been averaged where applicable.
- FR-INJ1 (Equation A-9 in Appendix A).

(Details on these models, including equations, are given in Appendix A.)

Only the first study from Sweden provides a model that predicts total crashes, and the UK models apply only to four-leg roundabouts. For the SWED-TOT1 model, both 70-km/h (44-mph) and 50-km/h (31-mph) speed limits were tested. With the use of a speed limit of 70 km/h [44 mph], testing determined that the model underpredicts crashes (see Table 12). Further testing with the 50-km/h [31-mph] speed limit revealed further underprediction of crashes.

The results of the statistical goodness-of-fit tests, which are also shown in Table 12, indicate that the mean prediction bias per site-year (MPB/site-yr) and mean absolute deviation per site-year (MAD/site-yr) are roughly of the same magnitude, if not greater, than the number of crashes per site-year. This statistic indicates that none of the models fit the data very well.

The existing models include other limitations:

- No model exists for roundabouts with more than four legs or more than two circulating lanes.
- Only one model predicts total crashes, and, by predicting a crash rate independent of traffic volume, it inherently assumes a linear relationship between traffic volumes and crashes.

Table 12. Goodness-of-fit tests of existing intersection-level models to U.S. data.

Model	SWED-TOT1	SWED-INJ1	SWED-INJ2	SWED-INJ3	UK-INJ1	UK-INJ2	UK-INJ3	FR-INJ1
	Total	Injury	Injury	Injury	Injury	Injury	Injury	Injury
No. Sites	78	78	78	78	20	20	61	85
MPB	–3.45	0.49	0.03	0.44	2.77	2.81	2.86	–0.28
MPB/site-yr	–1.18	0.17	0.01	0.15	1.01	1.02	1.00	–0.10
MAD	5.49	1.34	1.09	1.30	3.15	3.17	3.17	1.84
MAD/site-yr	1.88	0.46	0.37	0.44	1.14	1.15	1.11	0.64
MSPE	71.65	4.06	2.83	3.58	23.05	24.85	22.29	13.60
MSPE/site-yr^2	0.11	0.01	<0.01	0.01	0.15	0.16	0.04	0.02
AADT average (range)	15,539 (2,668–37,564)	15,539 (2,668–37,564)	15,539 (2,668–37,564)	15,539 (2,668–37,564)	21,050 (5,322–36,770)	21,050 (5,322–36,770)	16,434 (3,870–37,564)	15,908 (3,870–37,564)
Crashes/site-yr	2.63	0.24	0.24	0.24	0.29	0.29	0.36	0.41
Notes			Only applicable to urban roundabouts		Only applicable to four-leg roundabouts and where major and minor AADTs known separately		Only applicable to four-leg roundabouts	Applicable where entering AADT between 32,000 and 40,000

Legend: MPB = mean prediction bias; MAD = mean absolute deviation; MSPE = mean square prediction error; AADT = average annual daily traffic; yr = year

In light of these limitations and the poor goodness of fit to the U.S. data, the use of existing models to represent the U.S. data is an undesirable option for developing intersection-level prediction models for use in the United States. On this basis, the research team determined that new intersection-level models based on U.S. data needed to be calibrated to have models available for both total crashes and injury crashes at the various site types found in the United States.

Ability of Existing Non-U.S. Approach-Level and Other Disaggregate Safety Prediction Models to Represent U.S. Data

Similarly, to test the possibility of using the UK approach-level models directly on the U.S. database, U.S. data were applied to the existing approach-level and other disaggregate models from the UK to assess their goodness of fit. The UK model form is described in Appendix A.

Initially, the U.S. data developed for this project and crash data used to develop the UK models were compared (Table 13). As shown in the table,

- The U.S. data have a much higher percentage of exiting-circulating crashes;
- The UK data have a much higher percentage of crashes involving loss of control, although this percentage includes crashes in the circulating part of the roundabout, which the U.S. data does not; and
- The U.S. data have a much smaller proportion of pedestrian crashes.

Before testing the UK models, the research team removed from the model any variables for which the necessary data were unavailable by evaluating the estimated parameters at their mean values and adding this value as a multiplicative factor.

Table 14 shows the full UK models (for each crash type) and the mean value of each variable removed. Note that the U.S. data were applied to the more general form of the Maycock and Hall model (*4*). In the final Maycock and Hall model, some variables were omitted (e.g., sight distance and gradient category), and some coefficients were rounded compared to those used here.

Next, the full UK models were recalibrated against the U.S. data by testing them as specified, and then calculating a recalibration term defined as the ratio of observed crashes to predicted crashes. This recalibration term is simply added to the original model as a multiplicative factor. Table 15 describes which UK model was tested against which set of U.S. data for recalibration. Table 16 provides goodness-of-fit statistics for the UK models tested against the U.S. data for entering-circulating, approaching, and single-vehicle crashes.

In light of the limitations imposed by the differences in crash categories between the UK models and the U.S. data, the calibration of the UK models on only fatal and injury

Table 13. Comparison of disaggregated crash data in the U.S. and UK databases.

Crash Type	U.S. Data		Percentage in UK[1]	Notes
	Incidence	Percentage		
Entering-circulating	141	23%	43.3%	
Exiting-circulating	187	31%	14.5% (Defined as "other" in the UK)	Other crashes include exiting-circulating, circulating-circulating, etc.
Rear-end on approach lanes	187	31%	17.0% (Defined as "approach" in the UK)	Most approaching crashes in UK are rear-ends.
Loss of control on approach lanes	77	13%	20.1%	In UK, this type includes single-vehicle crashes on circulating part of roadway.
Pedestrian	5	1%	5.1%	
Bicyclist	8	1%	–	
Total	605	100%	100%	

[1]Only fatal plus injury crashes

Table 14. Approach-level models by crash type at UK roundabouts showing mean values for variables not in the U.S. data.

Crash Type	Model Term	Parameter Value	Mean Value	Term at Mean Value
Entering-Circulating Crashes (Crashes involving an entering and a circulating vehicle)				
L(constant)	*Lk*	-3.09		
L(entering flow)	LQ_e	0.65		
L(circulating flow)	LQ_c	0.36		
Entry path curvature	C_e	-40.3		
Entry width	*e*	0.16		
Approach width correction	*ev*	-0.009		
Ratio factor	*RF*	-1.0		
Percentage of motorcycles	*Pm*	0.21	2.24	1.60
Angle to next leg	*A*	-0.008		
Gradient category	*g*	0.09	-0.11	0.99
Approaching Crashes (Crashes between vehicles approaching the roundabout—mostly rear-ends)				
L(constant)	*Lk*	-4.71		
L(entering flow)	LQ_e	1.76		
Entry path curvature	C_e	20.7		
Reciprocal sight distance	$1/V_r$	-43.9	0.015	0.52
Entry width	*e*	-0.093		
Gradient category	*g*	-0.13	-0.11	1.01
Single-Vehicle Crashes (Crashes involving single vehicle anywhere in intersection)				
L(constant)	*Lk*	-4.71		
L(entering flow)	LQ_e	0.82		
Approach half-width	*v*	0.21		
Entry path curvature	C_e	23.7		
Approach curvature category	C_a	-0.17	0.05	0.99
Reciprocal sight distance	$1/V_r$	-33.0		
Other (non-pedestrian) Crashes (includes exiting-circulating, exiting-exiting, circulating, etc.)				
L(constant)	*Lk*	-5.69		
L(entering × circulating flow)	LQ_{ec}	0.73		
Percentage of motorcycles	*Pm*	0.21	2.24	1.60
Pedestrian Crashes				
L(constant)	*Lk*	-3.59		
L((entering + exiting vehicle flow) × pedestrian flow)	LQ_{exp}	0.53		

Source: Maycock and Hall (*4*)

Table 15. UK models matched with U.S. data for recalibration of approach-level and other disaggregate models.

UK Model Based on Fatal and Injury Crashes	U.S. Crashes Applied to Model
Entering-circulating: Crashes involving an entering and a circulating vehicle.	Entering-circulating: Crashes involving an entering and a circulating vehicle.
Approaching: Crashes between vehicles on the approach. Mostly rear-ends.	Rear-ends on approach lanes
Single Vehicle: Crashes involving single vehicle anywhere in junction.	Loss of control on approach lanes
Other (non-pedestrian): Crashes include exiting-circulating, exiting-exiting, circulating, etc.	Not attempted because this category is not compatible with U.S. data collected
Pedestrian	Not attempted because pedestrian flows unknown and only 5 total pedestrian crashes in database
Bicyclist – No UK Model	No model

crashes, and the relatively poor goodness of fit to the U.S. data as evidenced by the relatively high values of MAD/site-year (compared to the crashes/site-year) and the relatively high calibration factors, using the existing models to represent the U.S. data is not a desirable option for developing approach-level prediction models for use in the United States. On this basis, new approach-level models need to be calibrated based on U.S. data.

Models Calibrated for U.S. Data

This section presents the development of U.S. intersection-level and approach-level models. These new models are directly calibrated using the data assembled for this project and model forms that others have found successful for roundabout and general intersection modeling. See Appendix E for definitions of statistical terms.

Intersection-Level Prediction Models

Intersection-level safety prediction models were calibrated for total and injury crashes; the latter includes fatal and definite injury crashes and excludes possible injury and property damage only (PDO) crashes. To develop the models, a variety of variable sets were tested:

- AADT entering the intersection only
- AADT, number of legs, and number of lanes
- AADT, number of legs, number of lanes, and the ratio of central island diameter to inscribed circle diameter
- AADT, number of legs, number of lanes, and inscribed circle diameter
- AADT, number of legs, number of lanes, and central island diameter

Generalized linear modeling was used to estimate model coefficients using the software package SAS and an assumed negative binomial error distribution, all consistent with the state of research in developing these models.

Consistent with common practice, the models calibrated are of the following very general and flexible form:

$$Crashes/year = \exp(Intercept)\cdot AADT^{b_1}\exp(X_1+\cdots+X_n) \quad (3\text{-}1)$$

where

$AADT$ = average annual daily traffic entering the intersection

$X_1 \cdots X_n$ = independent variables other than AADT in the model equation

b_1 = calibration parameter

Table 16. Goodness-of-fit tests of the ability of UK entering-circulating, approaching, and single-vehicle models to represent U.S. data.

Measure	Crash Type		
	Entering-Circulating	Approaching	Single Vehicle
	UK	UK	UK
Number of legs	81	110	107
Calibration factor (observed/predicted)	1.82	3.83	1.29
MAD/site-yr	0.14	0.36	0.16
MSPE/site-yr^2	0.0004	0.0035	0.0006
Crashes/site-yr	0.32	0.57	0.55

Legend: MAD = mean absolute deviation; MSPE = mean square prediction error

Notes: The *number of legs* is the number of roundabout legs the data were recalibrated against. The *calibration factor* is the recalibration factor for the UK models calculated by dividing the sum of observed crashes by the sum of predicted crashes.

In selecting the recommended intersection-level models for total and injury crashes, the research team looked for low values of the dispersion parameter and statistical significance of the estimated variable coefficients. Tables 17 and 18 summarize these major considerations. The recommended models are the ones that achieve the lowest dispersion parameter values while having all variables significant at a level of at least 10 percent.

The analyses indicate that a model including AADT, number of legs, and number of lanes has the best fit to the data available for calibration. The research team also believes that, at the planning level, the inclusion of central island diameter and inscribed circle diameter was not appropriate, as a practitioner assuming values for these dimensions may introduce artificial error in the prediction. The final calibrated models are shown in Tables 19 and 20. SAS output including detailed statistics, such as standard errors of the estimated parameters, are presented in Appendix F.

It is important to reiterate that these models have been calibrated to the data available to this project. When using the models for a particular jurisdiction, they should be recalibrated using data for a sample of roundabouts in the jurisdiction. To do this, the local jurisdiction dataset is applied to the model provided in Equation 3-1. A calibration factor is calculated as the ratio of the sum of crashes actually recorded in the sample to the sum of the model predictions for individual roundabouts in the sample. The individual local jurisdiction calibration factor is then applied to Equation 3-1. At a minimum, data for at least 10 roundabouts with at least 60 crashes are needed to complete this calibration.

Approach-Level Crash Prediction Models

The general purpose of the approach-level models is to understand the impacts of geometric design decisions on various crash types. For example, as the designer evaluates different design options (e.g. entry width, entry radius, or central island diameter), he/she can assess the direction, if not the magnitude, of the safety consequence of the selection. These models are not intended as predictive models in the same sense that the intersection-level models are. However, if they are used for this purpose, it is stressed that a multiplier should be calibrated, as for the intersection-level models, to reflect local conditions.

Table 17. Comparison of intersection-level model results for total crashes.

Model	Variables	Significance of Variable Coefficients (10% level)	Dispersion Parameter
1	AADT	AADT significant.	1.4986
2	AADT, number of legs and number of lanes	All variables significant.	0.8986
3	AADT, number of legs, number of lanes, and ratio of central island diameter to inscribed circle diameter	Central island diameter/ inscribed circle diameter ratio is not significant; other variables are.	0.8348
4	AADT, number of legs, number of lanes, and inscribed circle diameter	Inscribed circle diameter is not significant; other variables are.	0.7792
5	AADT, number of legs, number of lanes, and central island diameter	Central island diameter is not significant; other variables are.	0.8408

Table 18. Comparison of intersection-level model results for fatal and injury crashes.

Model	Variables	Significance of Variable Coefficients (10% level)	Dispersion Parameter
1	AADT	AADT significant.	1.7262
2	AADT, number of legs, and number of lanes	All variables significant.	0.9459
3	AADT, number of legs, number of lanes, and ratio of central island diameter to inscribed circle diameter	AADT and central island diameter/inscribed circle diameter ratio are not significant; other variables are.	0.8714
4	AADT, number of legs, number of lanes, and inscribed circle diameter	AADT and inscribed circle diameter are not significant; other variables are.	0.6891
5	AADT, number of legs, number of lanes, and central island diameter	AADT and central island diameter are not significant; other variables are.	0.8894

Table 19. Intersection-level safety prediction model for total crashes.

Number of Circulating Lanes	Safety Performance Functions [Validity Ranges]		
	3 legs	4 legs	5 legs
1	$0.0011(AADT)^{0.7490}$ [4,000 to 31,000 AADT]	$0.0023(AADT)^{0.7490}$ [4,000 to 37,000 AADT]	$0.0049(AADT)^{0.7490}$ [4,000 to 18,000 AADT]
2	$0.0018(AADT)^{0.7490}$ [3,000 to 20,000 AADT]	$0.0038(AADT)^{0.7490}$ [2,000 to 35,000 AADT]	$0.0073(AADT)^{0.7490}$ [2,000 to 52,000 AADT]
3 or 4	Not In Dataset	$0.0126(AADT)^{0.7490}$ [25,000 to 59,000 AADT]	Not In Dataset
Dispersion factor, k=0.8986			

Table 20. Intersection-level safety prediction model for injury crashes.

Number of Circulating Lanes	Safety Performance Functions [Validity Ranges]		
	3 legs	4 legs	5 legs
1 or 2	$0.0008(AADT)^{0.5923}$ [3,000 to 31,000 AADT]	$0.0013(AADT)^{0.5923}$ [2,000 to 37,000 AADT]	$0.0029(AADT)^{0.5923}$ [2,000 to 52,000 AADT]
3 or 4	Not In Dataset	$0.0119(AADT)^{0.5923}$ [25,000 to 59,000 AADT]	Not In Dataset
Dispersion factor, k=0.9459			

The approach-level safety performance functions (SPFs) were developed for specific crash types: entering-circulating, exiting-circulating, and approaching. Due to the relatively small number of crashes being modeled, the SPFs were developed for total crashes only. Generalized linear modeling was applied to estimate model coefficients using the software package SAS and an assumed negative binomial error distribution, all consistent with the state of research in developing these models. These models are of the following form:

$$Crashes/year = \exp(Intercept)\cdot AADT_1^{b_1}\cdots\cdot AADT_m^{b_m} \cdot\exp(c_1X_1+\cdots+c_nX_n) \quad (3\text{-}2)$$

where

$AADT_1\cdots AADT_m$ = average annual daily traffic
$X_1\cdots X_n$ = independent variables other than AADT in the model equation
$b_1\cdots b_m$, $c_1\cdots c_n$ = calibration parameters

The variables tested include entry radius, entry width, central island diameter, approach half-width (referred to as approach width on Figure 9), circulating width, and others.

Tables 21, 22, and 23 present the candidate models for entering-circulating, exiting-circulating, and approaching crashes, respectively. These tables can be interpreted by applying the values in the tables to the model form given above. For example, Model 6 in Table 21 has the following equation form:

$$Crashes/year = \exp(-7.2158)\cdot(AADT_E)^{0.7018}\cdot(AADT_C)^{0.1321} \cdot\exp(0.0511e-0.0276\theta) \quad (3\text{-}3)$$

where

$AADT_E$ = entering AADT for the subject entry
$AADT_C$ = circulating AADT conflicting with the subject entry
e = entry width (ft)
θ = angle to next leg (degrees)

Table 21. Entering-circulating crash candidate models for total crashes.

Model No.	Dispersion	Intercept	Entering AADT	Circulating AADT	Entry Radius (ft)	Entry Width (ft)	Central Island Diameter (ft)	Angle to Next Leg (deg.)	1/Entry Path Radius (1/ft)
1	1.665	-13.2495	1.0585	0.3672					
2	1.664	-13.0434	0.9771	0.3088	0.0099				
3	1.495	-12.2601	0.9217	0.2900		0.0582	-0.0076		
4	1.514	-13.0579	1.0048	0.3142	0.0103		-0.0046		
5	1.302	-8.7613	0.9499	0.2687	0.0105			-0.0425	
6*	1.080	-7.2158	0.7018	0.1321		0.0511		-0.0276	
7	2.032	-8.9686	0.8322	0.1370					-138.096

*Recommended model

Table 22. Exiting-circulating crash candidate models for total crashes.

Model No.	Dispersion	Intercept	Exiting AADT	Circulating AADT	Inscribed Circle Diameter (ft)	Central Island Diameter (ft)	Circulating Width (ft)	1/ Circulating Path Radius (1/ft)	1/Exit Path Radius (1/ft)
1	6.131	-7.7145	0.3413	0.5172					
2*	2.769	-11.6805	0.2801	0.2530	0.0222		0.1107		
3	3.015	-11.2447	0.3227	0.3242		0.0137	0.1458		
4	3.317	-3.8095	0.2413	0.5626				372.8710	
5	4.430	-9.8334	0.6005	0.7471					-387.729

*Recommended model

Because of correlations among the variables in the individual candidate models and the small sample size of crashes, calibration of a model with more than a few variables was not possible. The candidate models presented for each crash type are quite close statistically. They all tend to contain logical variables with estimated effects in the expected direction. Appendix H contains the SAS output for each of these models.

To recommend the models with the best predictive power, the research team looked for the models with the lowest dispersion parameters while ensuring that the variables in the selected models and the direction of the indicated effects were logical. These recommended models are marked with an asterisk and shaded in Tables 21 through 23. On the basis of the dispersion parameter, models with AADT as the only explanatory variable, clearly, do not have the predictive power of models that contain at least one geometric variable. These AADT-only models are nevertheless presented in Tables 21 through 23 because they are intended for consideration for *Highway Safety Manual* (HSM)-type predictions that are discussed in Chapter 6.

The recommended models do not incorporate the wide array of geometric design features that engineers will be working with as the roundabout design is being developed. However, consistent with current prototype HSM procedures, the analysis does allow for the estimated coefficients for geometric features in recommended and other models to be considered in developing crash modification factors (CMFs) for use in the HSM. For example, the designer can use the AADT-only models (Model No. 1 given in Tables 21 through 23) and then identify the effect of a design change by applying the appropriate CMF (shown in Table 24). For example, to determine the effect of a unit change in entry width on entering-circulating crashes, the designer would first determine a base level of entering-circulating crashes using Model No. 1 given in Table 21 as follows:

$$Crashes/year = \exp(-13.2495) \cdot AADT_E^{1.0585} \cdot AADT_C^{0.3672} \tag{3-4}$$

The designer would then apply the implied CMF from Table 24 that relates a unit change in entry width to entering-circulating crashes: 1.0524. Caution is advised, however, because many of the variables are correlated, resulting in model-implied effects that may not reflect reality. Therefore, the correlations should be considered when determining which CMFs might be used in the HSM. To this end, a correlation matrix is provided as Table 25.

Although the number of entering-circulating, exiting-circulating, and approaching crashes predicted with the approach-level models can be added together to estimate the total number of crashes at a roundabout, the designer is advised to use the intersection-level model for the purposes of estimating the number of crashes at a roundabout. The intersection-level model is better for this purpose because (1) recalibrating the approach-level models for local conditions is more difficult than recalibrating the intersection-level models, and (2) the intersection-level models were developed specifically for such prediction while the approach-level models were developed to assess designs.

Table 23. Approaching crash candidate models for total crashes.

Model No.	Dispersion	Intercept	Entering AADT	Approach Half-Width (ft)
1	1.330	-5.6561	0.6036	
2*	1.289	-5.1527	0.4613	0.0301

*Recommended model

Development of Speed-Based Prediction Models Using U.S. Data

The concept of a speed-based model that relates safety performance to absolute speeds and/or relative speeds (speed consistency) was pursued with the hope of providing an intermediate link to both safety and operational performance. The rationale is that speed profiles are a manifestation of the driver's response to a design. Speed profiles are

Table 24. CMFs implied from candidate approach-level models for unit change in variable.

Variable	Entering-Circulating	Exiting-Circulating	Approaching
Entry Radius (ft)	0.9901 to 0.9896	–	–
Entry Width (ft)	1.0524 *	–	–
Approach Half-Width (ft)	–	–	1.0306 *
Inscribed Circle Diameter (ft)	–	1.0224 *	–
Central Island Diameter (ft)	0.9924 to 0.9954	1.0138	–
Circulating Width (ft)	–	1.1171 *	–
Angle to Next Leg (deg.)	0.9728 *	–	–

*CMF was derived from the recommended model.

Table 25. Correlation analysis of approach-level independent variables.

Parameter	Statistic	Inscribed circle diameter	Entry width	Approach half-width	Entry radius	Circulating width	Central island diameter	Angle to next leg	Entering AADT	Circulating AADT	Exiting AADT
Inscribed circle diameter	Pearson Correlation	1.000	0.653	0.611	0.399	0.689	0.946	-0.169	0.245	0.131	0.085
	Sig. (2-tailed)	.	0.000	0.000	0.000	0.000	0.000	0.050	0.005	0.193	0.394
	N	139	138	132	131	138	134	135	130	100	102
Entry width	Pearson Correlation	0.653	1.000	0.818	0.455	0.827	0.629	-0.219	0.416	0.136	0.300
	Sig. (2-tailed)	0.000	.	0.000	0.000	0.000	0.000	0.011	0.000	0.178	0.002
	N	138	138	131	130	138	133	134	129	100	102
Approach half-width	Pearson Correlation	0.611	0.818	1.000	0.187	0.698	0.597	-0.213	0.392	0.186	0.185
	Sig. (2-tailed)	0.000	0.000	.	0.037	0.000	0.000	0.014	0.000	0.073	0.073
	N	132	131	132	125	131	127	132	123	93	95
Entry radius	Pearson Correlation	0.399	0.455	0.187	1.000	0.327	0.336	0.150	-0.004	0.023	0.023
	Sig. (2-tailed)	0.000	0.000	0.037	.	0.000	0.000	0.093	0.969	0.821	0.821
	N	131	130	125	131	130	126	127	122	97	99
Circulating width	Pearson Correlation	0.689	0.827	0.698	0.327	1.000	0.658	-0.194	0.599	0.203	0.281
	Sig. (2-tailed)	0.000	0.000	0.000	0.000	.	0.000	0.025	0.000	0.042	0.004
	N	138	138	131	130	138	133	134	129	100	102
Central island diameter	Pearson Correlation	0.946	0.629	0.597	0.336	0.658	1.000	-0.234	0.310	0.072	0.021
	Sig. (2-tailed)	0.000	0.000	0.000	0.000	0.000	.	0.007	0.000	0.477	0.833
	N	134	133	127	126	133	134	130	125	100	102
Angle to next leg	Pearson Correlation	-0.169	-0.219	-0.213	0.150	-0.194	-0.234	1.000	-0.316	-0.124	-0.001
	Sig. (2-tailed)	0.050	0.011	0.014	0.093	0.025	0.007	.	0.000	0.228	0.991
	N	135	134	132	127	134	130	135	126	96	98
Entering AADT	Pearson Correlation	0.245	0.416	0.392	-0.004	0.599	0.310	-0.316	1.000	0.647	0.596
	Sig. (2-tailed)	0.005	0.000	0.000	0.969	0.000	0.000	0.000	.	0.000	0.000
	N	130	129	123	122	129	125	126	130	98	98
Circulating AADT	Pearson Correlation	0.131	0.136	0.186	0.023	0.203	0.072	-0.124	0.647	1.000	0.220
	Sig. (2-tailed)	0.193	0.178	0.073	0.821	0.042	0.477	0.228	0.000	.	0.028
	N	100	100	93	97	100	100	96	98	100	100
Exiting AADT	Pearson Correlation	0.085	0.300	0.185	0.023	0.281	0.021	-0.001	0.596	0.220	1.000
	Sig. (2-tailed)	0.394	0.002	0.073	0.821	0.004	0.833	0.991	0.000	0.028	.
	N	102	102	95	99	102	102	98	98	100	102

especially relevant to roundabouts for which it is widely believed that speed management is the key to how safe a roundabout is.

The models are of the following form:

$$Crashes/year = \exp(Intercept) \cdot AADT^{b} \cdot \exp(cX) \qquad (3\text{-}5)$$

where

$AADT$ = average annual daily traffic

X = independent speed-related variable

b, c = calibration parameters

Crashes were modeled with AADT and the observed speeds at various locations through the roundabout as independent variables. Speeds were measured upstream of the entry, at the entry point, at the exit point, and in front of the splitter islands at the entry and exit points. With the available data, only models for crashes between vehicles approaching the roundabout showed any distinct relationship to speed. Thirty-six legs had speed data and volume data suitable to calibrating a model for approaching crashes. Table 26 shows the results of this analysis. Appendix I presents the statistical results of the various models that were tested.

The models, on the whole, were deemed inadequate on the basis of the weak effects of the speed variables. However, the Australian experience (*5, 6*) and the one relatively successful model shown here indicate that a speed-based model approach is promising and that, with a more elaborate dataset, more can be made of it. At the moment, however, this approach is not recommended.

Before-After Analysis

The objective of the before-after analysis was to conduct a statistically defensible before-after study to estimate the safety benefits of installing roundabouts. While such studies have previously been done using U.S. data (*9, 10*), the goal was to build on those studies using a database that was richer in number of intersections and number of years of data, thus providing the ability to further disaggregate the results. In so doing, the hope was that insights could be gained into conditions that favor roundabout installation from a safety perspective by examining how the safety effect estimates vary with the following factors:

- Traffic volumes
- Type of control before (signal or stop)
- Crash history
- Number of legs
- Single-lane or multilane designs
- Setting (urban versus rural)

The empirical Bayes before-after procedure (*31*) was employed to properly account for regression-to-the-mean while normalizing, where possible, for differences in traffic volume between the before and after periods. The change in safety at a converted intersection for a given crash type is given by

$$\text{Change in safety} = B - A \qquad (3\text{-}6)$$

where

B = the expected number of crashes that would have occurred in the after period without the conversion

A = the number of reported crashes in the after period

B was estimated using the empirical Bayes procedure in which an SPF for the intersection type before roundabout conversion is used to first estimate the annual number of crashes (P) that would be expected at intersections with traffic volumes and other characteristics similar to the one being evaluated. The SPF crash estimate is then combined with the count of crashes (x) in the n years before conversion to obtain a site-specific estimate of the expected *annual* number of crashes (m) at the intersection before conversion. This estimate of m uses weights estimated from the mean and variance of the regression estimate as follows:

$$m = w_1 x + w_2 P \qquad (3\text{-}7)$$

where

m = expected site-specific annual number of crashes before conversion

Table 26. Speed-based approach candidate models.

Model No.	Overdispersion parameter	Intercept	Entering AADT	Speed Differential (mph)	Approach Speed (mph)
1	1.3683	-9.0059	0.8255	0.0622	
2*	1.3346	-9.9951	0.8609		0.0521

Legend: Speed Differential = difference between the speed of vehicles approaching the roundabout and the speed of entering vehicles

*Recommended model

$$w_1 = \frac{P}{\frac{1}{k} + nP}$$

x = count of crashes in the n years before conversion

$$w_2 = \frac{\frac{1}{k}}{\frac{1}{k} + nP}$$

P = prediction of annual number of crashes using SPF for intersection with similar characteristics

k = dispersion parameter for a given model, estimated from the SPF calibration process with the use of a maximum likelihood procedure

Factors then are applied to account for the length of the after period and differences in traffic volumes between the before and after periods. The result is an estimate of B. The procedure also produces an estimate of the variance of B. The significance of the difference ($B-A$) is established from this estimate of the variance of B and assuming, based on a Poisson distribution of counts, that

$$Var(A) = A \tag{3-8}$$

In the estimation of changes in crashes, the estimate of B is summed over all intersections in the converted group of interest (to obtain B_{sum}) and compared with the count of crashes during the after period in that group (A_{sum}). The variance of B is also summed over all conversions. The variance of the after period counts, A, assuming that these are Poisson distributed, is equal to the sum of the counts.

The estimate of safety effect, the Index of Effectiveness (θ), is estimated as

$$\theta = \frac{A_{sum} / B_{sum}}{1 + Var(B_{sum}) / B_{sum}^{2}} \tag{3-9}$$

The percentage change in crashes is equal to $100(1 - \theta)$; thus, a value of $\theta = 0.70$ indicates a 30% reduction in crashes.

The variance of θ is given by

$$Var(\theta) = \theta^2 \frac{\frac{Var(A_{sum})}{A_{sum}^{2}} + \frac{Var(B_{sum})}{B_{sum}^{2}}}{\left(1 + \frac{Var(B_{sum})}{B_{sum}^{2}}\right)^2} \tag{3-10}$$

Table 27 lists the base SPFs used as described previously. These data were taken from a variety of reliable sources because data were not collected for this purpose in this project. These base SPFs were recalibrated for use in the specific jurisdictions using data for the sample of roundabout conversions for the period immediately before conversion. Only the data in the 1 year immediately prior to roundabout construction were used for this purpose to guard against the possibility that a randomly high crash count in earlier years may have prompted the decision to install the roundabout and therefore provide functions that would overestimate safety performance. Examination of annual crash trends in the before periods indicated that this decision was justified.

The composite results are shown in Table 28, both in terms of percentage reduction in crashes and the index of effectiveness, θ. Injury crashes are defined as those involving definite injury or fatality. In other words, PDOs and possible injury are excluded. Results are shown separately for various logical groups for which sample sizes were large enough to facilitate a disaggregate analysis. The aggregate results for all sites are reasonably consistent with those from the IIHS and NYSDOT studies. The following conclusions can be drawn:

- **Control type before.** There are large and highly significant safety benefits of converting signalized and two-way-stop–controlled intersections to roundabouts. The benefits are larger for injury crashes than for all crash types combined. For the conversions from all-way-stop–controlled intersections, there was no apparent safety effect.
- **Number of lanes.** Disaggregation by number of lanes was possible for urban and suburban roundabouts that were controlled by two-way stops before conversion. The safety benefit was larger for single-lane roundabouts than for two-lane designs, for both urban and suburban settings. All rural roundabouts were single lane.
- **Setting.** The safety benefits for rural installations, which were all single lane, were larger than for urban and suburban single-lane roundabouts.
- **Additional insights.** Further disaggregate analysis provided the following insights:
  – The safety benefits appear to decrease with increasing AADT, irrespective of control type before conversion, number of lanes, and setting.
  – For various combinations of settings, control type before conversion, and number of lanes for which there were sufficiently large samples, there was no apparent relationship to inscribed or central island diameter.

Conclusion

The safety analysis described in this chapter results in a set of intersection-level prediction tools, approach-level prediction tools, and the most extensive disaggregation to date of U.S. crash performance before and after conversion to a roundabout. Further discussion of the significance and applicability of these findings can be found in Chapter 6.

Table 27. Base safety performance functions used in the empirical Bayes before-after analysis.

Setting	Previous Control	Number of Legs	Source of SPF Data	Model
Urban	Signal	4	Howard and Montgomery Counties, MD	Acc/yr = $\exp(-9.00)(AADT)^{1.029}$, k = 0.20 InjAcc/yr = $\exp(-10.43)(AADT)^{1.029}$, k = 0.20
Urban	Two-way stop	4	Howard and Montgomery Counties, MD	Acc/yr = $\exp(-1.62)(AADT)^{0.220}$, k = 0.45 InjAcc/yr = $\exp(-3.04)(AADT)^{0.220}$, k = 0.45
Urban	All-way stop	4	Minnesota – rural sites used due to lack of urban data	Acc/yr = $\exp(-12.972)(AADT)^{1.465}$, k = 0.50 InjAcc/yr = $\exp(-15.032)(AADT)^{1.493}$, k=1.67
Urban	Signal	3	California	Acc/yr = $\exp(-5.24)(AADT)^{0.580}$, k = 0.18 InjAcc/yr = $\exp(-6.51)(AADT)^{0.580}$, k = 0.18
Urban	Two-way stop	3	Howard and Montgomery Counties, MD	Acc/yr = $\exp(-2.22)(AADT)^{0.254}$, k = 0.36 InjAcc/yr = $\exp(-3.69)(AADT)^{0.254}$, k = 0.36
Urban	All-way stop	3	Minnesota – rural sites used due to lack of urban data	Acc/yr = $\exp(-12.972)(AADT)^{1.465}$, k = 0.50 InjAcc/yr = $\exp(-15.032)(AADT)^{1.493}$, k=1.67
Rural	Two-way stop	4	Minnesota	Acc/yr = $\exp(-8.6267)(AADT)^{0.952}$, k = 0.77 InjAcc/yr = $\exp(-8.733)(AADT)^{0.795}$, k = 1.25
Rural	All-way stop	4	Minnesota	Acc/yr = $\exp(-12.972)(AADT)^{1.465}$, k = 0.50 InjAcc/yr = $\exp(-15.032)(AADT)^{1.493}$, k=1.67

Legend: SPF = safety performance function; Acc/yr = total crashes per year; InjAcc/yr = fatal and injury crashes per year); AADT = average annual daily traffic entering the intersection; k = dispersion factor

Table 28. Results for before-after analysis by logical group.

Control Before	Sites	Setting	Lanes	Crashes recorded in after period		EB estimate of crashes expected without roundabouts		Index of Effectiveness θ (standard error) & Point Estimate of the Percentage Reduction in Crashes	
				All	Injury	All	Injury	All	Injury
All Sites	55	All	All	726	72	1122.0	296.1	0.646 (0.034) 35.4%	0.242 (0.032) 75.8%
Signalized	9	All	All	215	16	410.0	70.0	0.522 (0.049) 47.8%	0.223 (0.060) 77.7%
	4	Suburban	2	98	2	292.2	Too few	0.333 (0.044) 66.7%	Too few to estimate
	5	Urban	All	117	14	117.8	34.6	0.986 (0.120) 1.4%	0.399 (0.116) 60.1%
All-Way Stop	10	All	All	93	17	89.2	12.6	1.033 (0.146) -3.3%	1.282 (0.406) -28.2%
Two-Way Stop	36	All	All	418	39	747.6	213.2	0.558 (0.038) 44.2%	0.182 (0.032) 81.8%
	9	Rural	1	71	16	247.7	124.7	0.285 (0.040) 71.5%	0.127 (0.034) 87.3%
	17	Urban	All	102	6	142.7	31.6	0.710 (0.090) 29.0%	0.188 (0.079) 81.2%
	12		1	58	5	93.7	22.5	0.612 (0.101) 39.8%	0.217 (0.100) 80.3%
	5		2	44	1	48.9	Too few	0.884 (0.174) 11.6%	Too few to estimate
	10	Suburban	All	245	17	357.2	57.0	0.682 (0.067) 31.8%	0.290 (0.083) 71.0%
	4		1	17	5	77.1	21.8	0.218 (0.057) 78.2%	0.224 (0.104) 77.6%
	6		2	228	12	280.1	35.2	0.807 (0.091) 19.3%	0.320 (0.116) 68.0%
	27	Urban/ Suburban	All	347	23	499.9	88.6	0.692 (0.055) 30.8%	0.256 (0.060) 74.4%
	16		1	75	10	162.8	44.3	0.437 (0.060) 56.3%	0.223 (0.074) 77.7%
	11		2	272	13	329.0	44.3	0.821 (0.082) 17.9%	0.282 (0.093) 71.8%

CHAPTER 4

Operational Findings

This chapter presents a variety of modeling and evaluation tasks related to the operational performance of roundabouts in the United States. First, the results of the investigation of the ability of existing capacity and delay models to represent U.S. data are presented. Next, the chapter presents an analysis of gap acceptance parameters measured for U.S. sites. This section is followed by parametric analysis of factors, both geometric and flow related, that may be affecting driver behavior. Finally, the chapter presents the results of efforts to develop capacity models based on the data collected for this project, as well as a recommendation for level of service (LOS). Documentation for the data extraction, as well as supporting analyses and discussion for this chapter, can be found in Appendix J.

Assessment of Existing Capacity and Delay Models

The first step to determining an appropriate operational model for U.S. roundabouts is to determine how well existing operational models (both U.S. and international) represent U.S. conditions. Unlike safety analysis of roundabouts, which is in its relative infancy in the United States, the use of international capacity models for U.S. roundabouts is common practice. To be consistent with typical practice, these models have been initially examined without providing any local calibration. The following sections discuss the assessment of single-lane and multilane models separately.

Single-Lane Capacity Analysis

A variety of international and U.S. models (Australian, UK, German, French, Swiss, HCM 2000, and FHWA models, as described in Appendix B) were tested against the observed entry capacity. The predictions for a single-lane site with 10 or more minutes of queuing (MD07-E, Taneytown, Maryland) are illustrated in Figure 10. The *average error* (the average difference between the predicted and actual entry capacity) for each model by site is illustrated in Table 29. A positive average error implies that the model overpredicts the observed entry capacity. The root mean square error (RMSE) for each model across all sites is also illustrated. In general, a lower RMSE suggests a better prediction.

With the exception of two sites, all existing models predict higher capacities than observed at each site. For the WA08-S site (Kennewick, Washington), the German model most accurately predicts the WA08-S capacity, while the lower-bound HCM 2000 model tends to underpredict the capacity. In terms of the RMSE across all sites, the lower-bound HCM 2000, German, and FHWA models provide the best fit. The HCM 2000 and German models use default estimates of critical headway (historically referred to as "critical gap") and follow-up headway ("follow-up time") that are similar to the field predicted values, particularly at WA08-S, which has the lowest measured follow-up headway. Of the models tested, the uncalibrated French and UK models produce the largest error. Despite the large average error of the UK model, the predicted slope matches the data reasonably well. The slopes of the Australian and French predictions, dictated by the short critical headways, are higher than the general slope of the data.

Although the sample sizes are quite small, the international models clearly do not describe U.S. conditions well without further calibration. U.S. drivers appear to be either uncertain or less aggressive at roundabouts, and hence roundabouts currently appear to be less capacity efficient than the international models would suggest.

Multilane Capacity Analysis

The Australian, UK, German, French, Swiss, HCM 2000, and FHWA capacity models were also tested against the observed entry capacity for multilane entries. Predictions for an example site (WA09-E, Gig Harbor, Washington) are given in Figure 11.

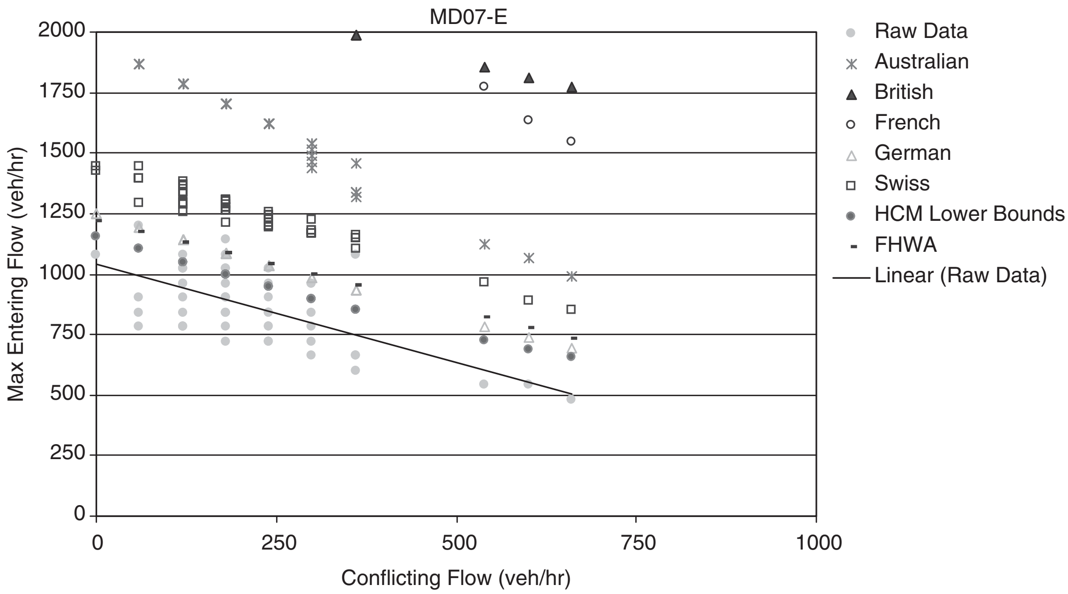

Figure 10. Example of uncalibrated entry capacity predictions at MD07-E.

For each site, only those data that are considered "plausible" for both entry lanes together, the left lane individually, and the right lane individually are included. (Data are "plausible" if the lane has a known queue occurring during the entire minute, either through visual verification of the video record or through examination of the critical headway and follow-up headway.) In general, very few plausible left-lane and total-entry data exist, suggesting that little queuing was observed in the left lane or simultaneously in both lanes. The sample size, average error, and RMSE are illustrated in Tables 30 and 31, respectively, for lane-based and approach-based models.

Several items are of note. First, the HCM 2000 and German models were included in the comparison, even though these were intended for use for single-lane roundabouts. Second,

Table 29. Capacity prediction error by model (uncalibrated), single-lane sites.

Site	*n*	Australian	UK	German	French	Swiss	HCM 2000 Upper	HCM 2000 Lower	FHWA
MD06-N	14	+485	+328	+172	+971	+199	+297	+80	+153
MD06-S	4	—[1]	+1228	+192	+1675	+382	+309	+105	+192
MD07-E	56	+535	+459	+240	+1024	+295	+362	+151	+227
ME01-E	42	+402	+507	+211	+729	+390	+328	+139	+229
ME01-N	1	—[1]	—[1]	—[1]	—[1]	—[1]	—[1]	—[1]	—[1]
MI01-E	8	+493	+933	+336	+1022	+511	+459	+282	+365
OR01-S	15	+292	+631	+122	+576	+268	+253	+85	+156
WA01-N	3	+226	+536	+246	—[1]	—[1]	+384	+223	+285
WA01-W	6	+91	+461	+117	+409	+257	+260	+101	+156
WA03-E	2	—[1]	—[1]	—[1]	—[1]	—[1]	—[1]	—[1]	—[1]
WA03-S	28	+416	+457	+260	+737	+338	+375	+185	+279
WA04-E	15	+531	+633	+134	+1091	+308	+249	+56	+149
WA04-N	85	+632	+814	+151	+1291	+352	+267	+67	+159
WA04-S	4	+411	+823	+136	+915	+317	+256	+78	+166
WA05-W	6	+450	+378	**–144**	+767	+209	+185	**–26**	+54
WA07-S	1	—[1]	—[1]	—[1]	—[1]	—[1]	—[1]	—[1]	—[1]
WA08-N	4	+609	+875	+267	+1080	+492	+389	+198	+281
WA08-S	24	+419	+460	+2	+815	+230	+119	**–77**	+11
RMSE across all sites	318	599	795	240	1191	376	331	183	240

Legend: *n* = number of observations; RMSE = root mean square error; negative errors (prediction less than observed) indicated in **bold**
[1]Insufficent observations

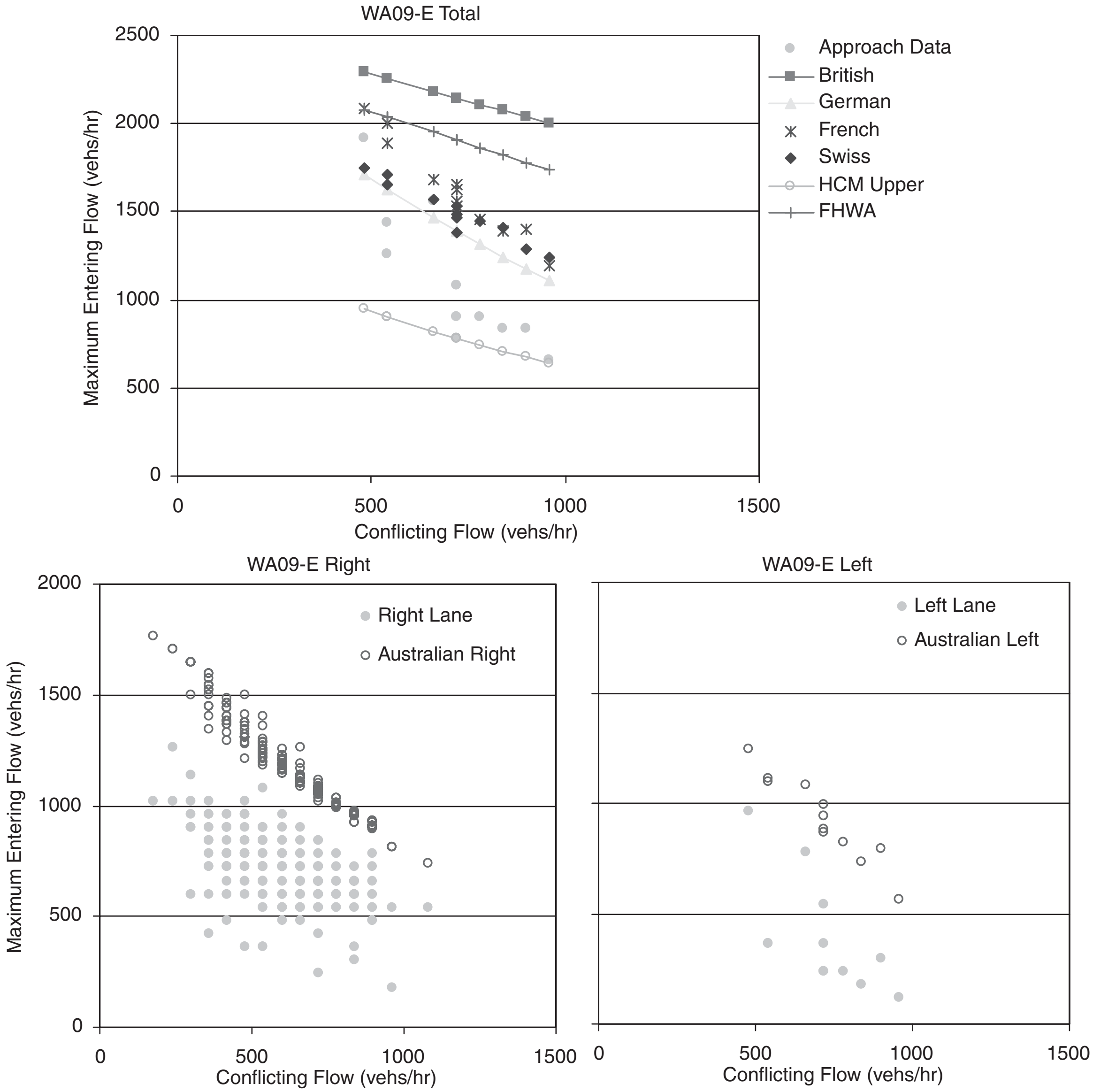

Figure 11. Example of uncalibrated model entry capacity predictions at WA09-E.

Table 30. Capacity prediction error by lane-based models (uncalibrated), multilane sites.

Site	*n*	Right Lane		Left Lane	
		n	Australian	*n*	Australian
MD04-E	36	22	+331	32	+318
MD05-NW	31	31	+173	—[1]	—[1]
MD05-W	3	3	+264	3	—[1]
VT03-E	16	16	+397	13	+411
VT03-S	20	20	+448	11	+631
VT03-W	83	82	+367	35	+491
WA09-E	194	193	+536	12	+585
RMSE across all sites	383	367	1054	106	473

Legend: *n* = number of observations; RMSE = root mean square error
[1]Not calculated

the MD04-E (Baltimore County, Maryland) and MD05-NW (Towson, Maryland) sites are not purely multilane for the purposes of this analysis: MD04-E has two entry lanes with only one conflicting lane, and MD05-NW has a single entry lane into two conflicting lanes.

With the exception of the upper- and lower-bound HCM 2000, all models overpredict the capacity. In terms of the RMSE, the upper-bound HCM 2000 and the German model provide the best fit. The Australian lane-based model and the French approach-based model produce the largest error, likely due to the short critical headways and follow-up headways inherent in those models. The UK model produces the highest average error at VT03-S (Brattleboro, Vermont), in part because of the effect of the large entry width on the estimated capacity.

Table 31. Capacity prediction error by approach-based models (uncalibrated), multilane sites.

Site	*n*	UK	German	French	Swiss	HCM Upper	HCM Lower	FHWA
MD04-E	21	+1059	+313	+532	+405	**–300**	**–462**	+822
MD05-NW	31	+876	+292	+654	+255	**–158**	**–315**	—[1]
MD05-W	3	+234	+61	+557	+279	+160	+45	—[1]
VT03-E	13	—[1]	+308	+618	+446	+84	**–25**	—[1]
VT03-S	11	+1243	+457	+872	+636	**–63**	**–213**	+1038
VT03-W	35	+1885	+345	+1338	+425	**298**	**–464**	+829
WA09-E	12	+1165	+405	+817	+543	**–172**	**–328**	+937
RMSE across all sites	126	459	375	818	490	271	367	953

Legend: *n* = number of observations; RMSE = root mean square error; negative errors (prediction less than observed) indicated in **bold**
[1]Not calculated

HCM Delay Analysis

The HCM 2000 control delay equation was tested against delay measurements obtained from the field. To maximize the use of available field data, the following assumptions were made:

- During queued minutes, the determination of the arrival flow at the back of queue was not always possible because of the limited field of view of the data collection equipment. In the extracted data, the arrival flow is equal to the entry flow, and hence the volume-to-capacity ratio is capped at 1.0. Queued-minute capacity is equal to the entry volume.
- For illustrative purposes, the capacity of non-queued minutes is defined by an exponential regression of queued minutes. The right-lane capacity regression is used to define the capacity in both the left and right lanes. The arrival volume during is equivalent to the entry volume.

In addition to the volume-to-capacity ratio, the HCM delay requires an analysis period, T. This parameter has a significant impact on the predicted delay at high volume-to-capacity ratios. While a value of T equals 0.25 h is typical, the volume and capacity conditions are assumed to be constant over the entire analysis period. The average delay will be higher than that predicted for an analysis period of 1 min. Because the field-delay data are presented in 1-min increments, a value of T equals 1 min (0.0167 h) is assumed.

As described previously, the field delay during periods of queuing likely excludes portions of the control delay. For this reason, two formulas for estimating delay have been tested: the HCM control delay equation, as documented in Appendix B, and what is termed here as the "HCM stopped delay" equation. The equation for HCM stopped delay used here is identical to that for control delay except that the "3600/c" and "+5" terms in the control delay equation have been omitted. Figure 12 illustrates the observed field delay and calculated HCM control and stopped delay for single-lane, 1-min observations. Figure 13 illustrates the HCM delay as a function of the field delay, with a 45-degree line representing an exact prediction. As can be seen in the figures, the variation in the field data is quite large, and both HCM predictions sit within the bounds of this variation. The correlations between the HCM control delay and the field delay, and the HCM stopped delay and field delay are 0.56 and 0.48, respectively. Considering the wide variation in the data, this correlation is quite good.

The 5-s adjustment for acceleration and deceleration included in the control delay equation seems to overpredict the delay, especially when the volume-to-capacity ratio is small. Drivers who are not in a queue and do not have to come to a complete stop at the yield line will not experience this additional delay. The research team recommends that this constant be modified to reflect this field experience. One method may be to adjust the constant using the volume-to-capacity ratio, as higher volume-to-capacity ratios indicate a higher potential for queuing, and hence acceleration and deceleration delay to and from a queued condition.

To determine whether delay prediction can be improved by reducing the overall variation in the delay data, the delay data can be aggregated into 5-min increments. This aggregation reduces the field data from 850 one-min observations to 126 five-min observations. However, this aggregation does not produce any improvement in the correlation and thus has not been carried forward.

RODEL and aaSIDRA Analysis

In addition to the individual capacity and delay models described previously, comparisons were made for single-lane roundabouts with two of the most common software packages used for roundabout analysis in the United States. These comparisons use the latest versions of each package available at the time of analysis: RODEL 1.0 and aaSIDRA 2.0 (note that at the time of this writing, aaSIDRA has since been updated to include additional calibration parameters).

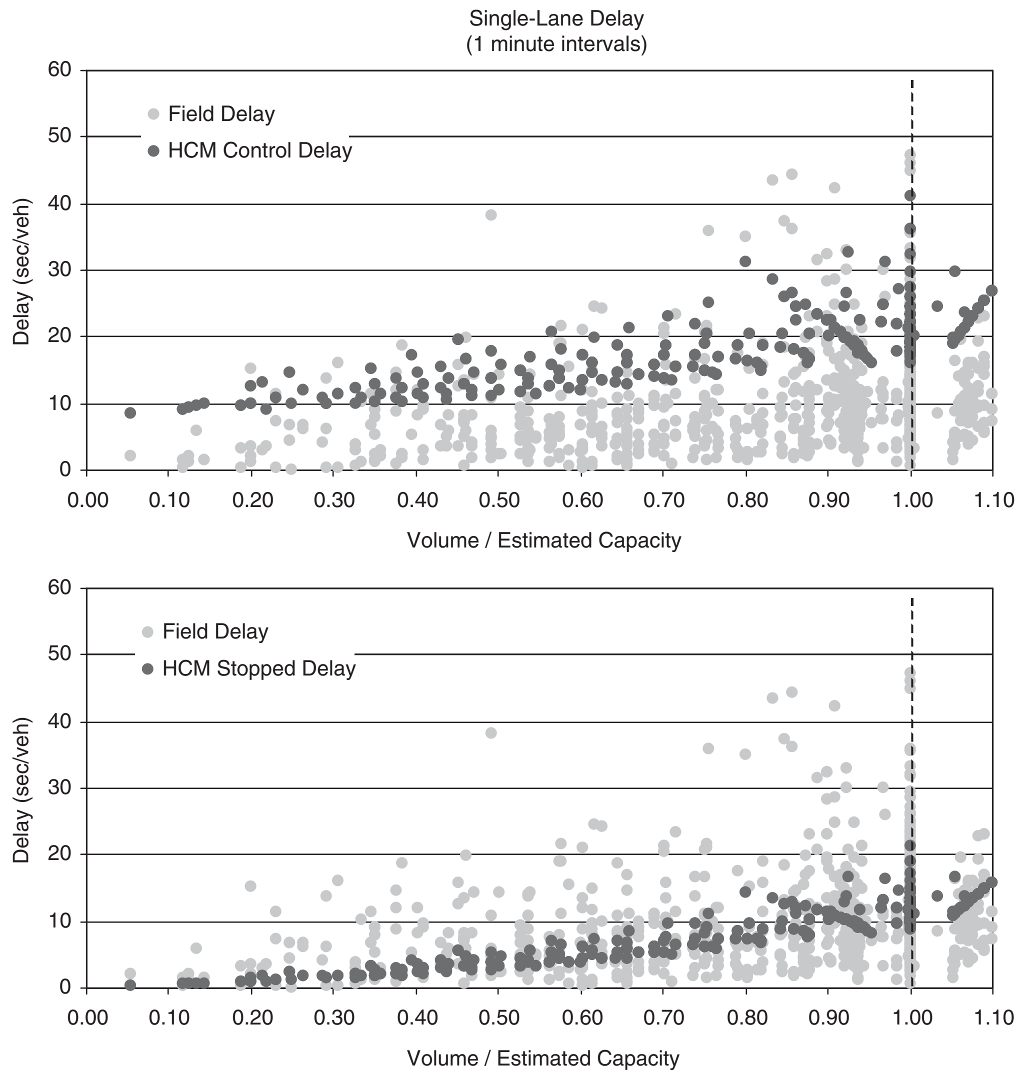

Figure 12. Minute-based HCM control delay, HCM stopped delay, and field delay at single-lane entries.

Approximate origin-destination matrices were developed for each roundabout during periods of continuous queuing, which have been used to estimate the turning movements and conflicting flow at each entry. In addition, flow rates were varied using a range of flow multipliers (10% to 500%) to enable capacity estimates across a broad range of conflicting flow rates for the given geometric conditions and turning movement proportions. Parameters such as the peak-hour factor and the percentage of trucks were not used.

RODEL, aaSIDRA, and the field-observed entry capacity have been plotted for each entry with more than 10 min of queued data. These plots are shown in Figure 14. As can be seen from the figure, both software models overestimate the field data. In the case of MD07-E (Taneytown, Maryland) and WA04-E (Port Orchard, Washington), the prediction is high because the entry width is unusually large. The slope of the RODEL capacity curve appears to match the field data more closely than the slope of the aaSIDRA capacity curve.

Figure 15 illustrates control delay as a function of the maximum entry flow plus the conflicting flow, with control delay plotted as a logarithm. As the capacity estimates from aaSIDRA and RODEL tend to be high relative to field data, the delay estimates tend to be correspondingly low.

Gap Acceptance Analysis

The operational evaluation also examined how individual vehicles accept and reject gaps at a roundabout entry. The reduction of these data and the calculation of the critical headway ("critical gap") and follow-up headway ("follow-up time") are described in subsequent sections.

Calculation of Critical Headway

The critical headway, t_c, is the minimum headway an entering driver would find acceptable. The driver rejects any headway

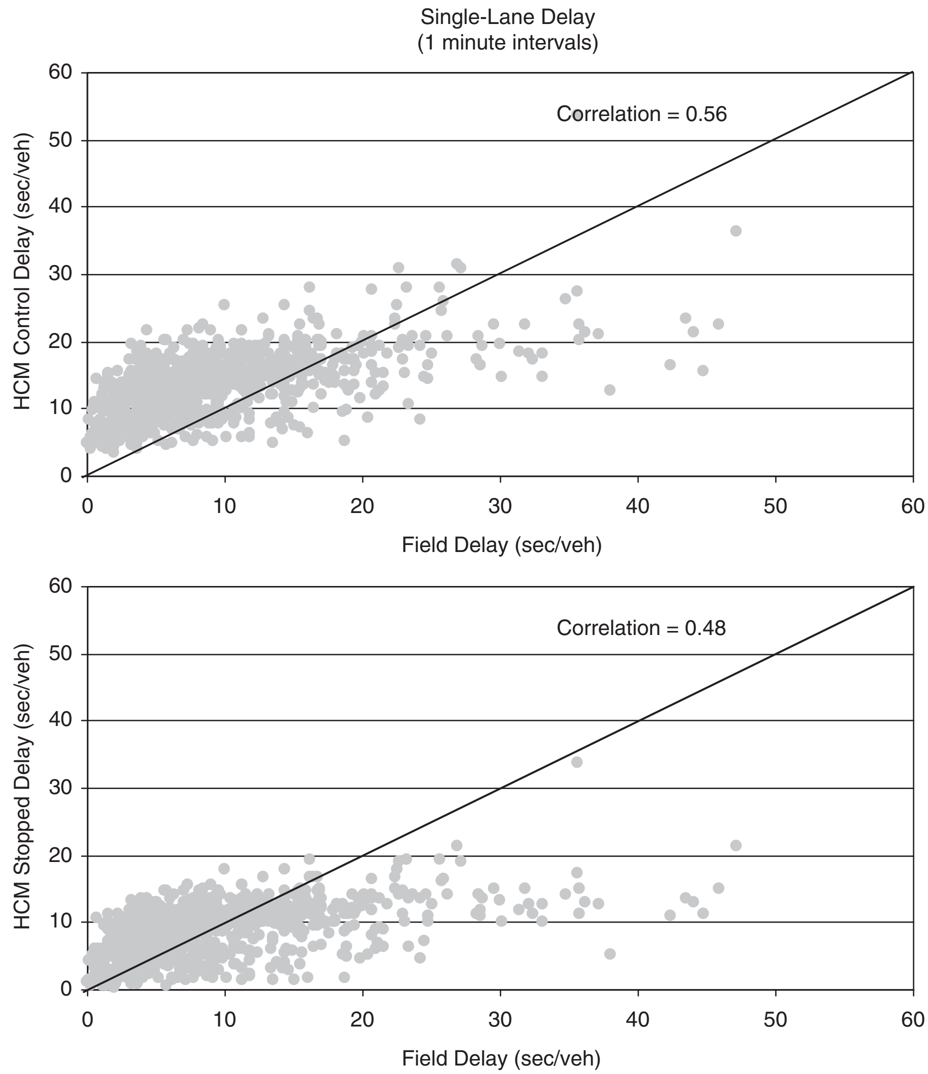

Figure 13. Comparison of control and stopped delay at single-lane entries.

less than the critical headway and accepts any headway greater than the critical headway. Hence, a driver's largest rejected headway will typically be less than the critical headway, and the accepted headway will be typically greater than the critical headway. Such theory assumes that the driver's behavior remains consistent. The concept is illustrated in Figure 16.

The critical headway was evaluated using the Maximum Likelihood Technique. This method is based on a driver's critical headway being larger than the largest rejected headway and smaller than the accepted headway. The probabilistic distribution for the critical headways is assumed to be log-normal.

Single-Lane Critical Headway

For each site, three methodologies for estimating critical headway were tested: (1) inclusion of all observations of gap acceptance, including accepted lags; (2) inclusion of only observations that contain a rejected gap; and (3) inclusion of only observations where queuing was observed during the entire minute and the driver rejected a gap. A lag is defined as the time from the arrival of the entering vehicle at the roundabout entry to the arrival of the next conflicting vehicle; this time is essentially a portion of the actual gap (as illustrated in Figure 17). The lags have been converted to gaps using an approximate follow-up headway.

The critical headway of each site and method is summarized in Table 32. The critical headway calculated using Method 1 is typically shorter than that from Method 2 because the average rejected gap is much shorter between the two methods (1 s versus 2 s), while the accepted gap is similar between the two methods. Method 2 uses a subset of the same data, and the larger the subset, the closer the estimate of the critical headway.

Although queued conditions were anticipated to result in more urgent acceptance of gaps, and hence a lower critical headway, such a phenomenon is not shown in the results. Some of the queued sample (Method 3) critical headways are less than and some are more than the Method 2 critical head-

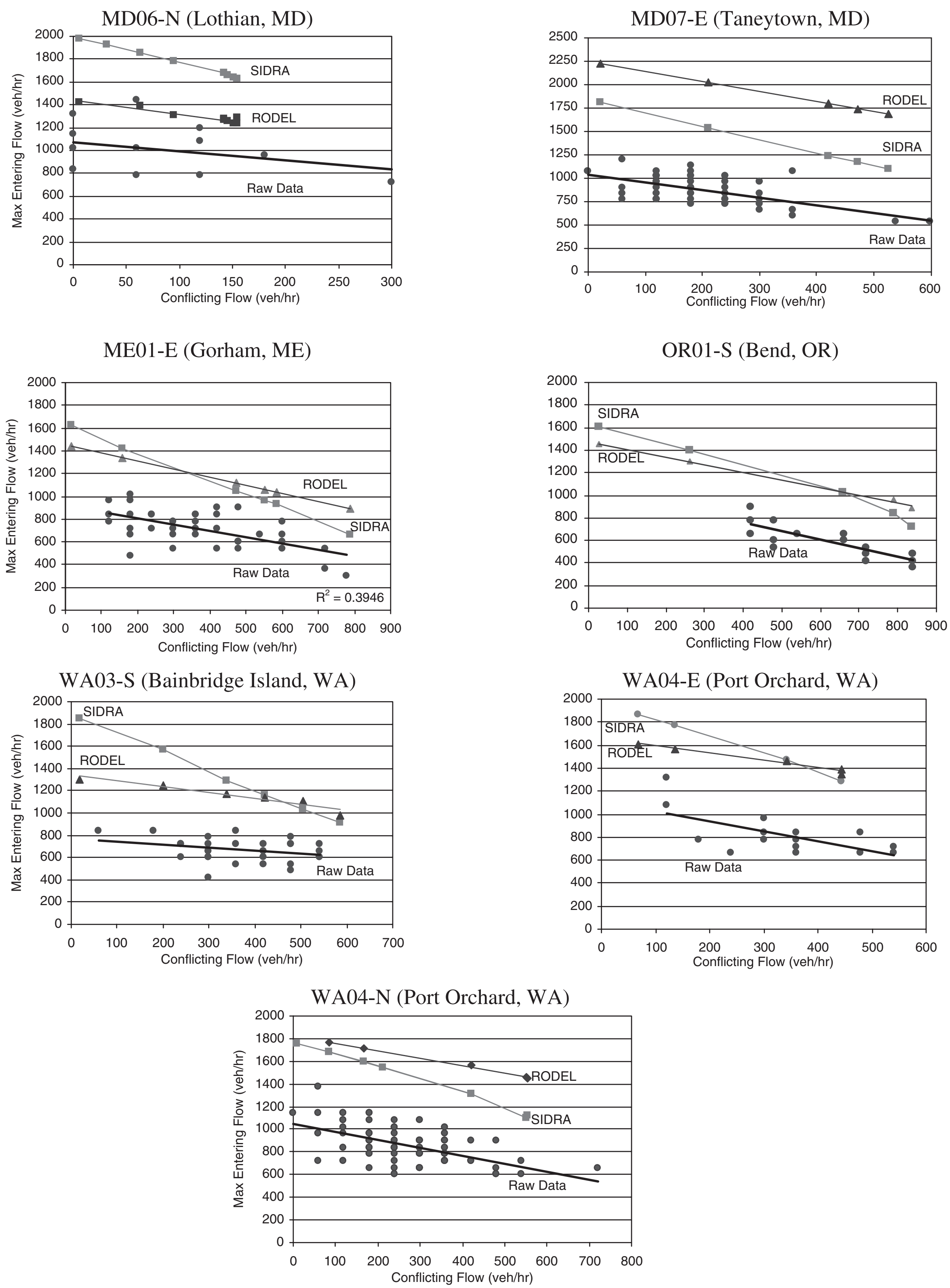

Figure 14. Field and software entry capacity as a function of the conflicting flow.

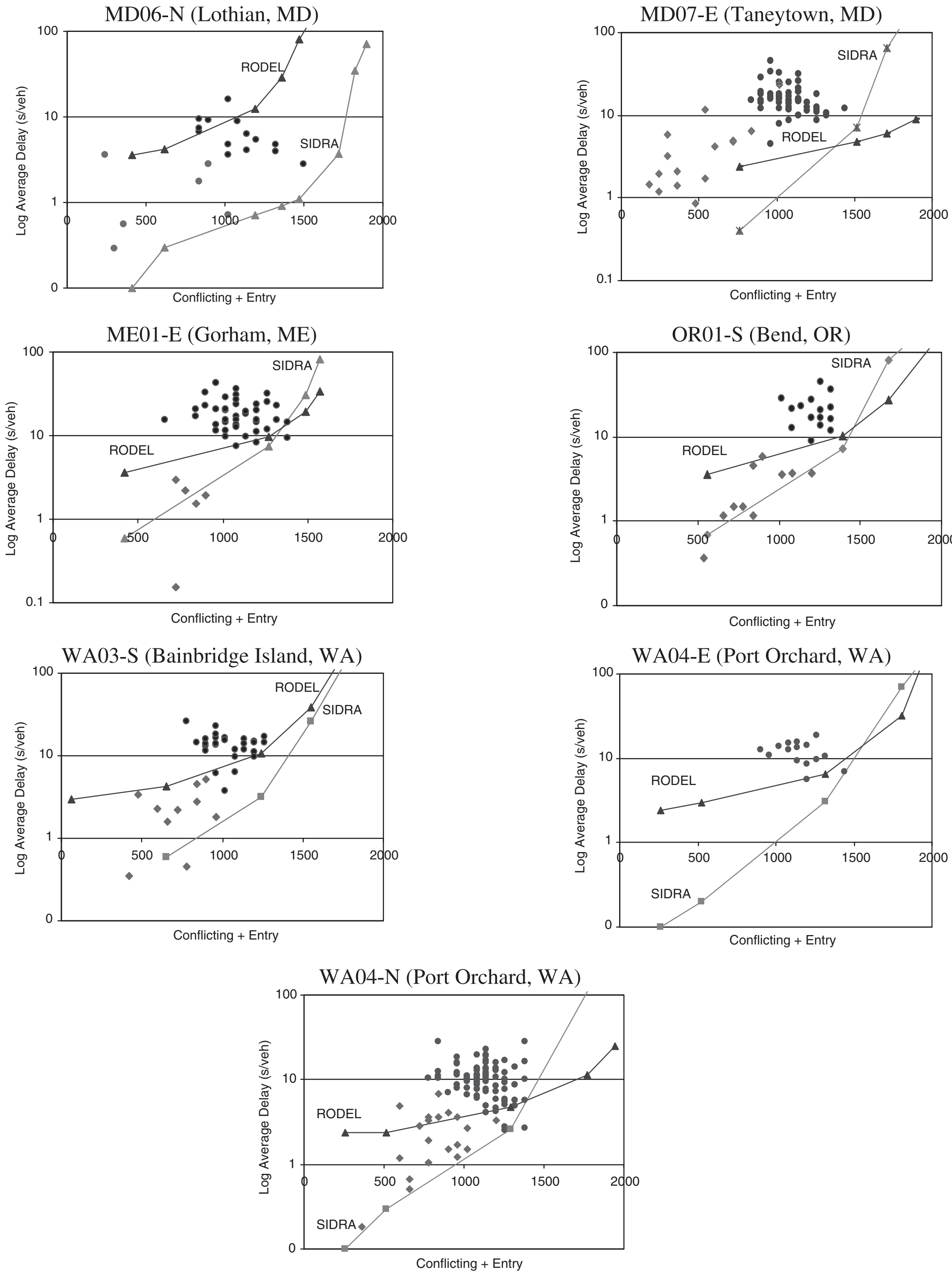

Figure 15. Log of average delay as a function of the maximum entry plus conflicting flow.

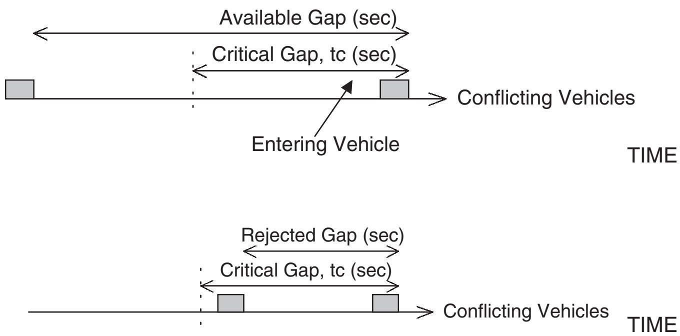

Figure 16. Concept of accepted and rejected gap.

ways. The difference between the average estimates is approximately 0.1 s.

As Method 1 assumes rejected gaps that are equal to zero, but not observed in the field, and Method 3 is similar to the results produced by Method 2 but has insufficient data to yield a critical headway estimate at some sites, Method 2 is the recommended methodology for use in this study.

The critical headway determined using Method 2 varies between 4.2 and 5.9 s, with a weighted average of 5.0 s. The average standard deviation is approximately 1.2 s, or 24%. Fewer than 1% of the drivers behaved inconsistently (accepting a gap smaller than previously rejected), so no additional adjustments for inconsistent drivers were required.

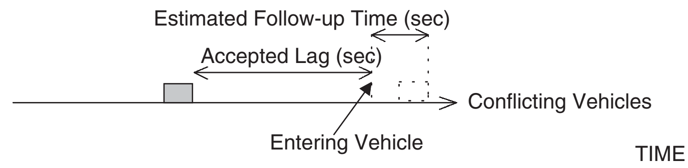

Figure 17. Concept of accepted lag.

Some sites have fewer than 50 critical headway observations. While the average critical headway of these sites may change with a larger sample size, the result is indicative of the average behavior of the site during those minutes when queuing was observed.

Multilane Critical Headway

For a multilane roundabout, the critical headway can be calculated using two techniques. One technique considers each entering lane and conflicting lane separately: the right entry lane uses the gaps in the outermost circulating lane (assuming that the entering vehicles in the right lane yield only to conflicting vehicles in the outer lane), and the left entry lane uses the combined gaps of the inner and outer

Table 32. Critical headway estimates, single-lane sites.

Site	Method 1[1]		Method 2[2]		Method 3[3]	
	n	t_c (std. dev.) (s)	n (% of Method 1)	t_c (std. dev.) (s)	n (% of Method 1)	t_c (std. dev.) (s)
MD06-N	733	4.2 (1.0)	32 (4%)	5.2 (1.8)	10 (1%)	5.5 (1.7)
MD06-S	76	4.9 (0.9)	38 (50%)	5.0 (1.0)	0 (0%)	—
MD07-E	1,602	4.3 (1.0)	174 (11%)	5.4 (1.5)	66 (4%)	5.4 (1.6)
ME01-E	820	4.2 (0.9)	198 (24%)	4.5 (1.0)	101 (12%)	4.6 (1.1)
ME01-N	98	5.1 (1.0)	51 (52%)	5.4 (1.2)	0 (0%)	—
MI01-E[5]	—[4]	—[4]	25 (—[4])	5.7 (0.9)	21 (84%)[6]	5.6 (0.8)
OR01-S	577	4.2 (1.1)	225 (39%)	4.7 (1.2)	66 (11%)	4.9 (1.2)
WA01-N	92	4.6 (0.7)	43 (47%)	4.7 (0.7)	0 (0%)	—
WA01-W	237	4.4 (1.8)	121 (51%)	4.4 (1.0)	0 (0%)	—
WA03-S	1,314	4.2 (1.1)	332 (25%)	5.0 (1.5)	66 (5%)	4.9 (0.9)
WA03-E	197	4.8 (2.1)	23 (12%)	5.3 (1.1)	0 (0%)	—
WA04-E	481	4.9 (1.0)	240 (50%)	5.3 (1.1)	97 (20%)	5.5 (1.3)
WA04-N	3,244	4.3 (0.9)	1,627 (50%)	5.2 (1.3)	152 (5%)	5.1 (1.3)
WA04-S	233	3.9 (0.7)	63 (27%)	4.2 (0.8)	0 (0%)	—
WA05-W	528	4.1 (1.0)	36 (7%)	5.9 (1.6)	0 (0%)	—
WA07-S	106	—[4]	22 (21%)	5.0 (0.8)	0 (0%)	—
WA08-N	582	—[4]	37 (6%)	5.8 (1.1)	0 (0%)	—
WA08-S	661	—[4]	60 (9%)	5.5 (1.5)	0 (0%)	—
Total	11,581		3,322 (29%)[7]		558 (5%)[7]	
Average		4.5 (1.0)		5.0 (1.2)[7]		5.1 (1.3)[7]

Legend: n = number of observations; t_c = critical headway; std. dev. = standard deviation
Notes:
[1]All observations of gap acceptance (lags and gaps)
[2]Observations that include a rejected gap
[3]Observations that include a rejected gap and occur in a minute with observed continuous queuing
[4]Not analyzed
[5]Site is single-lane entry against single circulatory stream, although roundabout is multilane
[6]Percentage of Method 2
[7]Totals exclude MI01-E

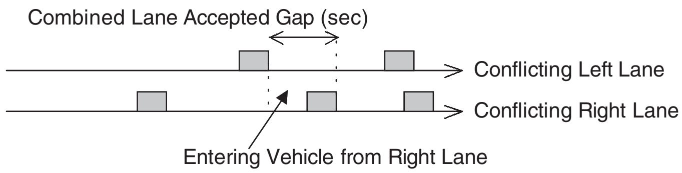

Figure 18. Concept of combined lane gaps.

conflicting lanes. An alternative technique estimates the critical headway for the entire approach, combining the entering lanes and conflicting lanes into single entering and conflicting streams, respectively.

For the purpose of calibrating the existing capacity models, the critical headway should be calculated with the technique used to develop those models. Troutbeck's critical headway research (used within aaSIDRA) is a lane-based model that considers the combined conflicting lane gaps (see Appendix B). For the right entry lane, this approach assumes that all conflicting vehicles have an influence on the entering drivers' behavior, which will be true in some cases and generally conservative. If a vehicle in the right entry lane enters at the same time as a vehicle is circulating in the inner conflicting lane, the defined accepted gap may be quite small. This event is illustrated in Figure 18.

The critical headways for the multilane-site data were determined using observations conforming to Methods 2 and 3 (described previously in "Single-Lane Critical Headway") and are presented in Table 33. Some of the queued sample (Method 3) estimated critical headways are less than and some are more than the Method 2 critical headways. The difference between the average critical headways is 0.1 s in the right lane and 0.2 s in the left lane.

The critical headways determined using Method 2 vary between 3.4 and 4.9 s in the right lane and 4.2 and 5.5 s in the left lane. It is interesting to note that the MD04-E (Baltimore County, Maryland) critical headways are longer than those observed at other multilane sites. Unlike the other sites with multilane entries used in this analysis, MD04-E has only one conflicting lane, which may explain the similarities of the critical headways to those observed at single-lane sites.

The average standard deviation is approximately 1.6 s, or 35%. Some sites have less than 50 critical headway observations for individual lanes. While the average critical headway of each site may change with a larger sample size, the result is indicative of the average behavior of the site during those minutes when queuing was observed.

Calculation of Follow-Up Headway

The follow-up headway, t_f, is defined as the headway maintained by two consecutive entering vehicles using the same gap in the conflicting stream. The entering vehicles must be in a queue. The follow-up headway may also be determined from observation of two consecutive vehicles entering the same lag.

An example of the time stamp vehicle data is shown in Table 34. The follow-up headway is the difference between the entry departure times of vehicles using the same gap. Vehicles using the same gap will have the same opposing vehicle time.

Table 33. Critical headway estimates, multilane sites.

Site	Method 2[1]				Method 3[2]			
	n		t_c (std. dev.) (s)		n (% of Method 2)		t_c (std. dev.) (s)	
	Left Lane	Right Lane	Left Lane	Right Lane	Left Lane	Right Lane	Left Lane	Right Lane
MD04-E	468	307	5.5 (2.6)	4.9 (2.1)	95 (20%)	62 (20%)	5.5 (2.5)	4.5 (3.8)
MD05-NW	275	—[3]	4.2 (2.3)	—[3]	126 (46%)	—[3]	4.1 (2.4)	—[3]
MD05-W	17	35	4.3 (1.6)	3.4 (1.2)	10 (59%)	16 (46%)	3.7 (1.2)	3.2 (1.0)
WA09-E	99	813	4.2 (2.2)	4.1 (1.6)	0 (0%)	629 (77%)	—[4]	4.1 (1.9)
VT03-W	237	604	4.4 (1.4)	4.2 (1.3)	114 (48%)	126 (21%)	4.2 (1.4)	4.1 (1.2)
VT03-E	100	115	4.3 (0.9)	4.0 (1.2)	30 (30%)	48 (42%)	5.0 (0.7)	4.5 (1.4)
VT03-S	73	182	5.0 (1.4)	4.4 (1.4)	10 (14%)	68 (37%)	5.1 (1.0)	4.5 (1.4)
Total	1,269	2,056			385 (30%)	949 (46%)		
	3,325				1,334 (40%)			
Average			4.8 (2.1)	4.3 (1.5)			4.6 (1.9)	4.2 (1.9)
			4.5 (1.7)				4.3 (1.9)	

Legend: n = number of observations; t_c = critical headway; std. dev. = standard deviation
Notes:
[1]Observations that include a rejected gap
[2]Observations that include a rejected gap and occur in a minute with observed continuous queuing
[3]Short flare in right lane received little use; entry effectively functioned as single-lane entry
[4]Insufficient observations

Table 34. Example of follow-up headway data and calculations.

Veh#	Entry Arrival Time Stamp	Entry Departure Time Stamp	Rejected Lag	Accepted Lag	Rejected Gap	Accepted Gap	Opposing Vehicle Time Stamp	Follow up Headway	Comments
1	01:11:23.2	01:11:24.5		00:12.1			01:11:35.3		
2	01:11:27.5	01:11:28.7		00:07.8			01:11:35.3	00:04.2	Same lag as #1
3	01:11:30.4	01:11:30.7		00:04.9			01:11:35.3	00:02.0	Same lag as #1
4	01:11:32.7	01:11:33.0		00:02.6			01:11:35.3	00:02.3	Same lag as #1
5	01:11:49.6	01:11:58.8	00:02.0		00:04.7	00:11.2	01:12:07.5		
6	01:12:00.6	01:12:00.9		00:06.9			01:12:07.5	00:02.2	Same gap as #5
7	01:12:03.0	01:12:03.6		00:04.5			01:12:07.5	00:02.7	Same gap as #5
8	01:12:06.0	01:12:14.2			00:03.3		01:12:18.0		Two rejected gaps before accepted gap
	01:12:06.0	01:12:14.2	00:01.5		00:02.4	00:07.2	01:12:20.4		
9	01:12:16.6	01:12:16.9		00:03.9			01:12:20.4	00:02.7	Same gap as #8
10	01:12:18.3	01:12:21.5	00:02.1			00:10.2	01:12:30.6		
11	01:12:23.7	01:12:24.0		00:06.9			01:12:30.6	00:02.5	Same gap as #10

The opposing vehicle time is calculated based on the accepted lag or gap:

- Using the accepted lag, the opposing vehicle time is the sum of the entry arrival time plus the accepted lag.
- Using the accepted gap, the opposing vehicle time is the sum of the entry arrival time plus the total rejected gaps and lag.

Approximately 40% of the extracted data occur within a full minute of queuing. Some of the remaining data contain valid follow-up headways observed in partial minutes of queuing. As a number of sites have limited full minutes of queuing, the time the next vehicle takes to move into entry position (move-up time) has been used to test the presence of a queue. Establishing a move-up time threshold allows valid data points within a portion of each minute of data to be used, thus expanding the overall database.

Single-Lane Follow-Up Headway

Figure 19 illustrates the frequency of the move-up and follow-up headway at WA04-N (Port Orchard, Washington) observed during periods of visually verified queues. Very few queues were observed beyond a move-up time of 6 s. Approximately 4% of the queued data exceed a move-up time of 6 s, and 22% of the queued data exceed a move-up time of 4 s. Applying a move-up threshold of 6 s to all of the extracted WA04-N data yields a similar move-up and follow-up headway distribution.

The follow-up headway for each lane has been calculated assuming that a move-up time less than 6 s indicates a queued condition. The queued and estimated follow-up headways are illustrated in Table 35. The difference between the follow-up headways of these two methodologies is approximately 0.2 s.

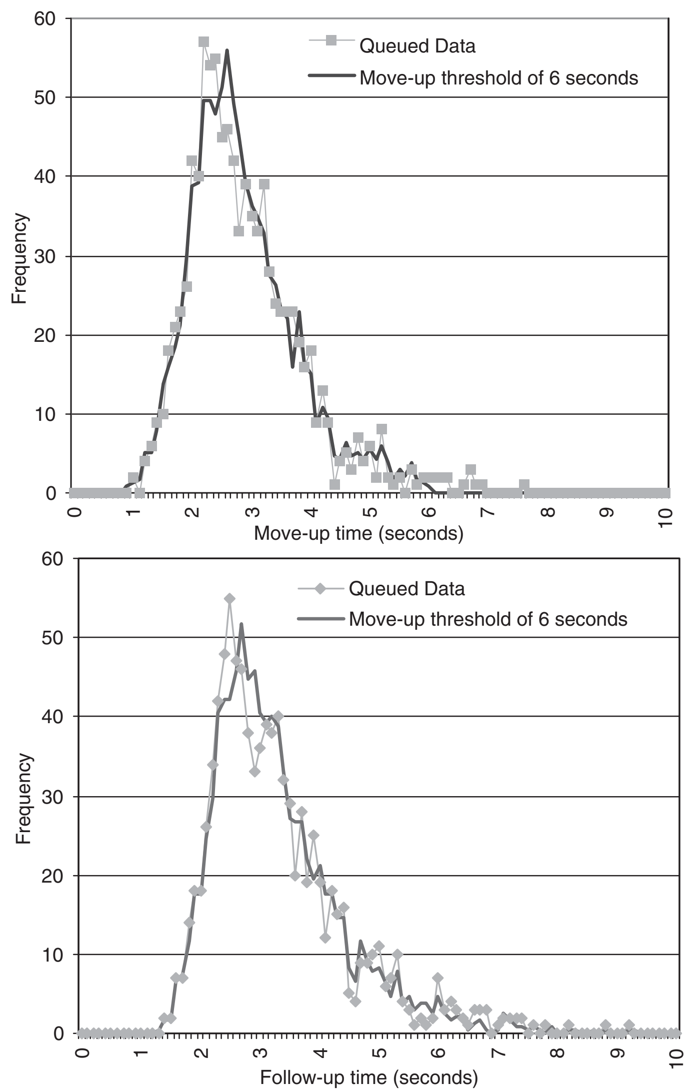

Figure 19. Move-up and follow-up time frequency (WA04-N).

Table 35. Follow-up headway estimates, single-lane sites.

Site	Queued		Move-up Time < 6 s	
	n	t_f (std. dev.) (s)	n	t_f (std. dev.) (s)
MD06-N	219	3.3 (1.3)	637	3.2 (1.1)
MD06-S	—[1]	—[1]	28	3.5 (1.3)
MD07-E	660	3.4 (1.1)	1,225	3.3 (1.1)
ME01-E	286	3.5 (1.2)	522	3.4 (1.1)
ME01-N	—[1]	—[1]	39	4.3 (1.5)
MI01-E[2]	24	3.7 (1.5)	41	3.5 (1.4)
OR01-S	86	3.0 (0.8)	262	3.1 (1.0)
WA01-N	—[1]	—[1]	33	3.4 (1.1)
WA01-W	—[1]	—[1]	86	3.3 (1.1)
WA03-E	—[1]	—[1]	126	3.8 (1.2)
WA03-S	199	3.7 (1.3)	753	3.6 (1.2)
WA04-E	140	3.1 (1.1)	334	3.1 (1.4)
WA04-N	952	3.3 (1.3)	2,282	3.2 (1.2)
WA04-S	—[1]	—[1]	120	3.1 (1.0)
WA05-W	103	3.2 (1.1)	453	3.1 (1.0)
WA07-S	—[1]	—[1]	80	2.9 (1.1)
WA08-N	—[1]	—[1]	400	2.9 (1.1)
WA08-S	327	2.6 (1.5)	438	2.6 (0.9)
Total	2,996		7,859	
Average		3.4 (1.2)		3.2 (1.1)

Legend: n = number of observations; t_f = follow-up headway; std. dev. = standard deviation

Notes:

[1]Insufficient observations

[2]Site is single-lane entry against single circulatory stream, although roundabout is multilane

Despite efforts to increase the dataset, a number of the sites have fewer than 100 follow-up headway observations. While the average follow-up headway of the entry lane may change with a larger sample size, the result is indicative of the average behavior of the entry queued minutes.

The smallest follow-up headway of 2.6 s was observed at WA08-S (Kennewick, Washington), which reflects the typically high maximum entry flow observations. The driver behavior at this site is likely impacted by the large student driver population. The largest follow-up headway of 4.3 s was observed at ME01-N (Gorham, Maine), which reflects the typically low maximum entry flow observations.

Multilane Follow-Up Headway

Table 36 summarizes the estimated follow-up headways for visually verified minutes of queuing and those for a move-up threshold of 6 s. Using a move-up threshold of 6 s, the follow-up headways in the right lane vary between 2.8 and 4.4 s. The left-lane follow-up headways are longer on average and vary between 3.1 and 4.7 s. VT03-S (Brattleboro, Vermont) has the lowest estimated follow-up headway of 2.8 s, which is reflected by a typically high maximum entering flow. Interestingly, the right lane behaves as two lanes (at times), which would appear as a very short follow-up headway.

The weighted average single-lane and multilane follow-up headways are 3.2 s and 3.1 s, respectively. Under follow-up conditions, the driver behavior at a single-lane or multilane roundabout is similar.

Summary

Table 37 illustrates the weighted average field data for critical headway and follow-up headway, along with various default parameters used in the international models tested. As can be seen from the table, the HCM 2000 and German models have similar follow-up headways and lower estimates of the

Table 36. Follow-up headway estimates, multilane sites.

Site	Queued				Move-up Time < 6 s			
	n		t_f (std. dev.) (s)		n		t_f (std. dev.) (s)	
	Left Lane	Right Lane	Left Lane	Right Lane	Left Lane	Right Lane	Left Lane	Right Lane
MD04-E	293	108	2.9 (1.0)	3.3 (1.7)	1,792	648	3.1 (1.1)	3.1 (1.5)
MD05-NW	125	—[2]	3.1 (1.0)	—[2]	315	—[2]	3.3 (1.2)	—[2]
MD05-W	—[1]	2	—[1]	4.4 (2.3)	6	2	4.7 (2.4)	4.4 (2.3)
VT03-E	28	44	3.5 (2.8)	3.4 (1.3)	73	104	3.2 (1.1)	3.1 (1.1)
VT03-S	8	159	3.4 (2.0)	2.8 (1.1)	85	478	3.4 (1.2)	2.8 (0.8)
VT03-W	91	592	3.5 (1.9)	3.2 (1.5)	180	1,340	3.3 (1.1)	3.1 (1.2)
WA09-E	39	1,249	5.0 (3.9)	2.9 (1.1)	28	1,773	3.5 (1.5)	3.0 (1.1)
Total	584	2,154			2,479	4,345		
	2,738				6,824			
Average			3.1 (1.4)	3.0 (1.2)			3.2 (1.1)	3.0 (1.2)
			3.1 (1.3)				3.1 (1.1)	

Legend: n = number of observations; t_f = follow-up headway; std. dev. = standard deviation

Notes:

[1]Insufficient observations

[2]Short flare in right lane received little use; entry effectively functioned as single-lane entry

Table 37. Summary of critical and follow-up headways.

Model	1 Lane		2 Lane	
	Follow-up headway, t_f (s)	Critical headway, t_c (s)	Follow-up headway, t_f (s)	Critical headway, t_c (s)
Field Measurements:				
Approach	2.6–4.3 (3.2)	4.2–5.9 (5.1)	N/A	N/A
Right Lane	N/A	N/A	2.7–4.4 (3.1)	3.4–4.9 (4.2)
Left Lane	N/A	N/A	3.1–4.7 (3.4)	4.2–5.5 (4.5)
Model Parameters:				
HCM 2000	3.1	4.6	N/A	N/A
German	3.2	4.4	3.2	4.4
French	2.1	N/A	2.1	N/A
Australian:				
Dominant Lane	1.8-2.7 (2.2)	1.4-4.9 (2.9)	1.8–2.8 (2.2)	1.6-4.1 (2.9)
Subdominant Lane	N/A	N/A	2.2–4.0 (3.1)	

critical headway. The Australian predictions of the critical headway and follow-up headway vary based on the conflicting flow, number of lanes, diameter, and entry width. At multilane sites, the follow-up headway is also a function of the *dominant* and *subdominant* arrival flows. The dominant entry lane is defined as the lane with the largest arrival flow. The predicted gap parameters used in the various single-lane and multilane models are generally smaller than the field-observed values. In addition, the field data appear to support the concept of dominant and subdominant lanes with respect to follow-up headways. In most cases, the right lane is dominant with a shorter follow-up headway than the left lane; however, MD04-E (Baltimore County, Maryland) is dominant in the left lane with a correspondingly shorter follow-up headway in the left lane. Additional sites with dominant left-lane arrival flow should be collected to validate the concept.

Parametric Analysis

One of the most important elements of the analysis contained in this chapter is the assessment of the influence of geometric and flow parameters on the capacity of roundabouts in the United States. The primary purpose for this analysis is to develop an understanding of the factors that have the most influence on U.S. roundabout capacity. Secondarily, it helps to assess the overall ability of various international models to represent U.S. data.

Influence of Geometry on Macroscopic Capacity

A number of international models use the geometry to modify capacity estimates. The Australian model suggests that the diameter influences the follow-up headway, while the entry width influences the critical headway. The French model uses the exiting path geometry and splitter width to adjust the influence of exiting vehicles on the capacity. Given that some entering vehicles tend to yield to exiting vehicles, the splitter island width may play a role in providing separation between these movements. The Swiss model accounts for a similar condition. The y-intercept of the UK model varies based on a combination of entry width, approach half-width, and effective flare length, modified by the entry angle and entry radius. Geometry such as the entry angle and entry radius may influence the speed in which the follow-up headway can be performed. The slope of the UK model is governed by the diameter, entry width, and flare length. Entry width has a significant influence on the capacity; the entry capacity increases by adding increments of width (the number of lanes present is implicit in the entry width and is not modeled directly).

The influence of geometry on the single-lane entry capacity has been analyzed using multiple linear regression. Some models use linear capacity relationships, including the UK and Swiss models. The UK model, however, assumes nonlinear relationships between the capacity and geometry, which are used to describe changes in the intercept and slope. For example, the shape of the relationship between the entry radius and entry capacity is logarithmic. For small values of the entry radius, less than 10 m (33 ft), the UK model suggests a significant and negative impact on the entry capacity. These relationships have been developed using a very large database comprising both field data and test-track data (see Appendix B). Local data provide the opportunity to examine the entry capacity as a function of the geometry on a total of 18 single-lane entries. Plots of the data have been prepared to support the identification of any non-linear relationships.

The following independent flow and geometric parameters were tested based on their inclusion in the UK and Australian capacity models:

q_c = Circulating traffic flow (veh/h)
D = Inscribed circle diameter of the roundabout (m)
e = Width of the entry at the edge of the circulating roadway (m)
Δe = Width of the flare of the entry = $e - v$ (m)
v = approach half-width (m)

l' = Effective flare length (m)
r = Entry radius (m)
ϕ = Entry angle (deg)

The geometry for each site is detailed in Tables 38 and 39 for single-lane and multilane roundabouts, respectively. The geometric parameters identified above are summarized as follows:

- Inscribed circle diameter: 31.7 to 58.5 m (104 to 192 ft) at single-lane sites; 27.4 to 75.6 m (90 to 248 ft) at multilane sites
- Entry width: 4.0 to 12.2 m (13 to 40 ft)
- Approach half-width: 3.0 to 8.5 m (10 to 28 ft)
- Effective flare length: 0 to 25 m (0 to 82 ft);
- Entry radius: 7.5 to 38.0 m (25 to 125 ft) at single-lane sites; 9.5 to 37.5 m (31 to 123 ft) at multilane sites

Table 38. Geometric parameters used for analysis, single-lane sites.

Site	Inscribed circle diameter, D	Entry width, e	Approach half-width, v	Effective flare length, l'	Entry radius, r	Entry angle, ϕ	Circulating width	Adjacent exit width	Adjacent departure width	Adjacent exit radius	Truck apron width	Central island radius	Splitter island width	Splitter island length
	m	m	m	m	m	°	m	m	m	m	m	m	m	m
MD06-N	36.6	4.6	3.7	10.1	18.3	20.0	6.1	5.5	3.7	18.3	3.7	9.1	4.0	5.5
MD06-S	36.6	4.6	3.7	5.8	18.3	19.0	6.1	5.5	3.7	18.3	3.7	9.1	4.6	5.5
MD07-E	44.5	7.6	8.2	0.0	21.3	38.0	9.8	8.2	3.7	24.4	3.0	10.4	2.7	8.2
ME01-E	33.0	4.5	5.0	0.0	38.0	18.0	7.5	5.5	4.0	1.0	8.0	3.5	3.5	5.5
ME01-N	33.0	4.5	5.5	0.0	7.5	50.0	3.5	5.0	4.5	20.0	7.5	3.5	4.0	5.0
MI01-E	36.0	5.6	4.5	25.0	24.0	18.0	9.5	8.4	4.5	∞	0.0	8.9	5.0	8.4
OR01-S	58.5	5.2	5.5	0.0	19.8	32.0	6.1	5.2	5.5	12.8	6.1	11.9	5.8	5.2
WA01-N	38.1	4.3	4.3	0.0	19.4	16.5	6.1	—[1]	—[1]	—[1]	3.7	9.3	6.1	—[1]
WA01-W	38.1	4.3	4.0	0.0	22.9	2.5	6.1	5.5	4.0	30.5	3.7	9.3	4.6	5.5
WA03-E	31.7	4.9	3.4	4.6	10.7	14.0	6.1	5.3	3.5	10.7	3.0	6.7	3.8	5.3
WA03-S	31.7	4.6	3.7	6.1	10.7	14.0	6.1	5.5	3.7	10.7	3.0	6.7	3.8	5.5
WA04-E	50.0	6.1	6.1	0.0	12.2	53.0	11.3	6.1	7.3	42.7	1.5	12.2	7.9	6.1
WA04-N	50.0	6.4	6.1	0.0	26.8	48.0	11.3	6.7	5.5	14.3	1.5	12.2	8.5	6.7
WA04-S	50.0	6.1	6.1	0.0	18.8	34.0	11.3	6.7	5.2	12.2	1.5	12.2	7.6	6.7
WA05-W	34.7	4.6	3.4	0.0	21.3	3.0	4.9	4.9	4.3	7.9	5.3	7.2	2.4	4.9
WA07-S	36.0	4.0	3.0	15.0	21.0	17.0	4.9	5.5	3.6	34.4	6.0	7.1	4.0	5.5
WA08-N	45.7	5.5	5.8	0.0	28.0	13.0	6.7	5.8	5.2	42.7	3.0	13.7	8.2	5.8
WA08-S	45.7	5.5	4.3	3.0	33.5	9.0	6.7	6.1	4.3	21.3	3.0	13.7	8.8	6.1
Maximum	*58.5*	*7.6*	*8.2*	*25.0*	*38.0*	*53.0*	*11.3*	*8.4*	*7.3*	∞	*8.0*	*13.7*	*8.8*	*8.4*
Minimum	*31.7*	*4.0*	*3.0*	*0.0*	*7.5*	*2.5*	*3.5*	*4.9*	*3.5*	*1.0*	*0.0*	*3.5*	*2.4*	*0.0*

[1]Approach is off-ramp with no adjacent exit.

Table 39. Geometric parameters used for analysis, multilane sites.

Site	Inscribed circle diameter, D	Entry width, e	Approach half-width, v	Effective flare length, l'	Entry radius, r	Entry angle, ϕ	Circulating width	Adjacent exit width	Adjacent departure width	Adjacent exit radius	Truck apron width	Central island radius	Splitter island width	Splitter island length
	m	m	m	m	m	°	m	m	m	m	m	m	m	m
MD04-E	27.4	7.6	7.3	6.7	25.0	12.0	5.8	4.9	3.7	8.4	1.5	15.2	2.3	4.9
MD05-NW	42.7	4.6	6.4	0.0	15.2	34.0	12.2	8.5	6.4	12.5	1.5	18.3	2.7	8.5
MD05-W	42.7	7.0	7.0	0.0	9.5	16.5	11.9	7.6	7.6	6.4	1.5	18.9	1.8	7.6
VT03-E	53.0	8.5	8.5	0.0	26.3	20.0	8.5	12.5	8.5	37.5	2.0	32.0	3.5	12.5
VT03-S	53.0	12.2	8.5	25.0	37.5	19.0	8.5	10.0	8.5	30.0	2.0	32.0	3.5	10.0
VT03-W	53.0	8.5	8.5	0.0	30.0	19.0	8.5	10.0	8.5	30.0	2.0	32.0	3.5	10.0
WA09-E	75.6	9.1	7.3	16.8	10.7	27.0	9.8	10.1	7.3	15.2	3.0	50.0	4.3	10.1
Maximum	*75.6*	*12.2*	*8.5*	*25.0*	*37.5*	*34.0*	*12.2*	*12.5*	*8.5*	*37.5*	*3.0*	*50.0*	*4.3*	*12.5*
Minimum	*27.4*	*4.6*	*6.4*	*0.0*	*9.5*	*12.0*	*5.8*	*4.9*	*3.7*	*6.4*	*1.5*	*15.2*	*1.8*	*4.9*

- Entry angle: 2.5 to 53.0 degrees at single-lane sites; 12.0 to 34.0 degrees at multilane sites.

To assess predictive quality, two attributes of each parameter have been determined: *correlation* and *contribution*. Correlation illustrates the strength of the linear relationship between the capacity and each of the parameters. Contribution is used to assess the usefulness of each parameter in the linear prediction of the capacity.

Standardized partial correlations of the entry capacity and each of the geometric parameters are depicted in Figure 20. Because of the minute-by-minute variation in the capacity, a linear correlation between the entry capacity and each of the geometric parameters cannot be seen by inspection of the graphs.

For each of the regression parameters, the slope coefficient and their significance of the contribution and correlation are shown in Table 40. The confidence level is equal to one minus

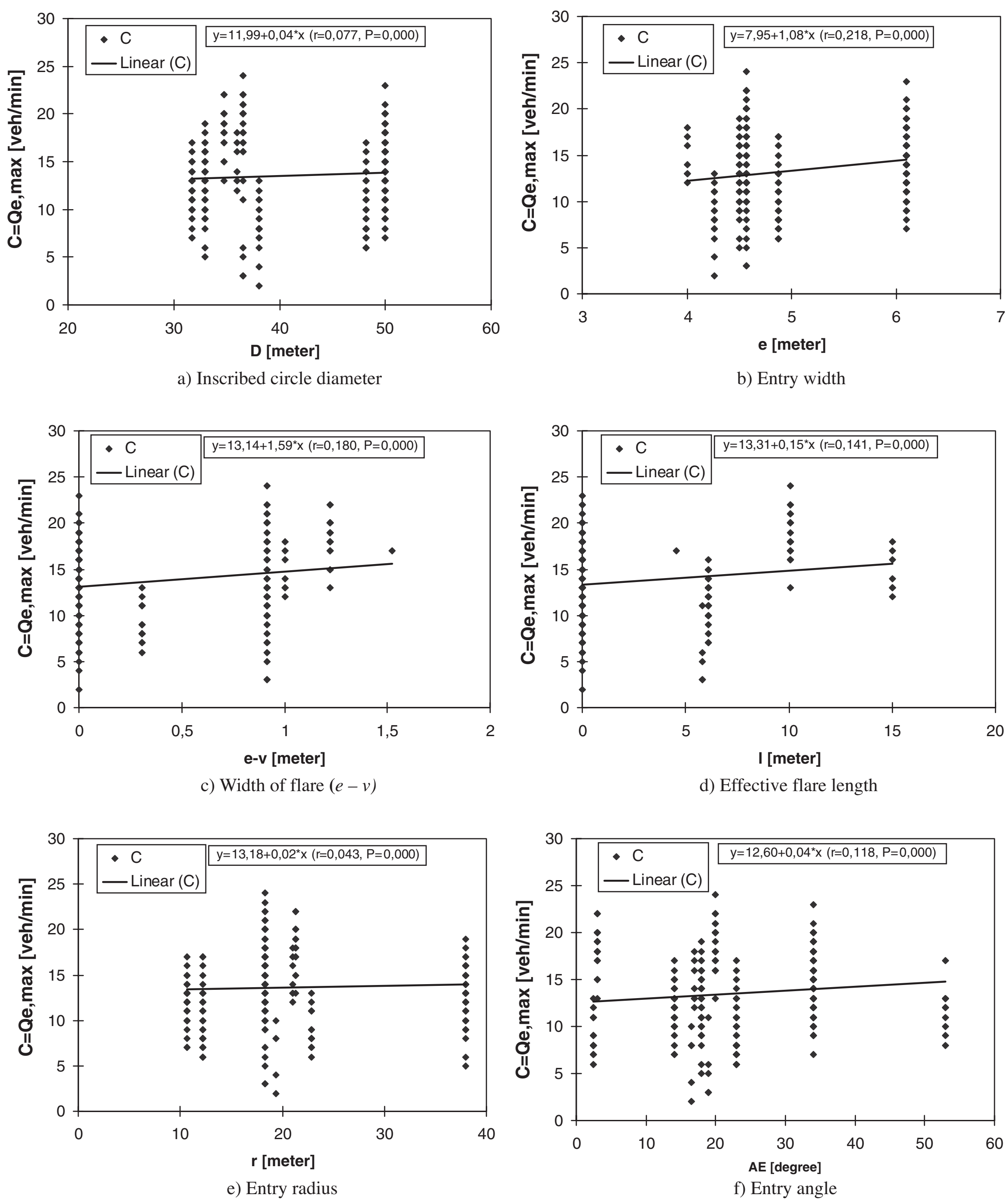

Figure 20. Partial correlations of entry capacity to geometric parameters.

Table 40. Results of the multiple linear regression analysis for single-lane entries.

Parameter	Non-Standardized Coefficient	Standardized Coefficient	Contribution Significance to Entry Capacity	Correlation Significance to Entry Capacity
	B	β (beta)	$1-P$	$1-P_c$
Y-intercept	11.212	—	0.005	—
Conflicting flow, Q_c	–0.831	–0.741	0.000	0.000
Diameter, D	0.114	0.233	0.001	0.075
Entry width, e	1.006	0.204	0.752	0.000
Width of flare, Δe (= $e - v$)	–0.740	–0.208	0.002	0.000
Effective flare length, l'	0.088	0.081	0.616	0.004
Entry radius, r	0.077	0.164	0.000	0.211
Entry angle, ϕ	–0.034	–0.099	0.896	0.013

Table 41. Summary of the significance of various parameters for single-lane entries.

Parameter	Contribution Significance to Entry Capacity		Correlation Significance to Entry Capacity		Total Significance to Entry Capacity	
	P=0.95	P=0.99	P_c=0.95	P_c=0.99	P=0.95 P_c=0.95	P=0.99 P_c=0.99
Conflicting flow, Q_c	Yes	Yes	Yes	Yes	Yes	Yes
Diameter, D	Yes	Yes	No	No	No	No
Entry width, e	No	No	Yes	Yes	No	No
Width of flare, Δe (= $e - v$)	Yes	Yes	Yes	Yes	Yes	Yes
Effective flare length, l'	No	No	Yes	Yes	No	No
Entry radius, r	Yes	Yes	No	No	No	No
Entry angle, ϕ	No	No	Yes	No	No	No

the significance. For example, the inscribed diameter, D, contributes to the entry capacity with a confidence level, P, of 0.999 ($1 - 0.001$), and D is correlated to the entry capacity with a confidence level, P_c, of 0.925 ($1 - 0.075$).

Assuming confidence-level thresholds of 0.95 and 0.99, the results have been summarized in a yes/no format in Table 41. In addition to the conflicting flow, the additional width created by any flare present, $\Delta e = e - v$, is significant for both the contribution and correlation with a confidence level, $P = P_c$, of 0.99. However, additional tests—including partial linear regression of the same seven parameters, multiple linear regression with a reduced number of parameters, and pair-wise multiple linear regression and correlation analysis (the conflicting flow paired step-wise with each of the seven parameters)—do not yield any parameters with a confidence level of 0.95. The significant variation in the entry capacity on a minute-to-minute basis for a given entry (such that the geometry is fixed) makes drawing conclusive relationships between capacity and geometry difficult.

Influence of Flow Parameters and Geometry on Driver Behavior

The critical headway and follow-up headway provide an indication of the driver behavior at a given entry. The follow-up headway represents the driver behavior when there are no conflicting vehicles. Its inverse is equivalent to the y-intercept of the relationship between entry capacity and conflicting flow. The critical headway represents the driver behavior during periods of conflicting flow, which typically impacts the slope of the entry-capacity/conflicting-flow relationship. For a given conflicting flow, a short critical headway will predict higher capacity estimates compared to a long critical headway.

Factors influencing the average driver behavior at sites may include the proportion of heavy vehicles, exiting and conflicting flow, and geometry. The influence of these parameters was examined using a simple correlation analysis, which measures how strong the linear relationship is between two variables. The correlation value varies between -1 and $+1$. A larger correlation value indicates a more linear relationship; a negative value implies a negative slope. A value near zero indicates the absence of a linear relationship but is not evidence of the lack of a strong non-linear relationship. Plots of the data were prepared to support the identification of any non-linear relationships.

The correlation results for the percentage of heavy vehicles, conflicting flow, exiting flow, and geometry are summarized in Table 42. The average gap parameters for the right lane of a multilane entry were used. Some of the critical headway and follow-up headway estimates are based on limited data and therefore were removed from the analysis.

Table 42. Correlation of flow and geometric factors to driver behavior.

Parameter	Correlation to t_c (All Sites)	Correlation to t_f (All Sites)	Correlation to t_f (Single Lane)
% Trucks (Entering)	**–0.52**	+0.07	+0.28
Conflicting Flow	**–0.63**	–0.15	+0.12
Exiting Flow	–0.34	–0.08	**+0.56**
Diameter	–0.37	–0.46	**–0.50**
Entry Lane Width	+0.37	–0.08	–0.07
Approach Half-Width	+0.07	–0.02	–0.10
Effective Flare Length	–0.13	–0.14	+0.18
Radius	–0.12	–0.41	–0.44
Entry Angle	+0.18	+0.03	–0.01
Splitter Width	–0.09	–0.30	**–0.53**

Legend: t_c = critical headway; t_f = follow-up headway
Note: Absolute values of correlations greater than or equal to 0.50 are indicated in **bold**

The critical headway and the percentage of heavy vehicles in the entering flow have a moderate negative correlation (−0.52). As illustrated in Figure 21, the critical headway decreases as the percentage of heavy vehicles increases. This finding is not intuitive (one would expect critical headway to increase with increasing heavy vehicles) but has not been explored further because of the limited amount of data with higher percentages of heavy vehicles.

The critical headway and conflicting flow have a moderate negative correlation (−0.63). Critical headway as a function of the conflicting flow is illustrated in Figure 22. The critical headway tends to decrease with increasing conflicting flow. Troutbeck notes the same observation in the Australian model (see Appendix B). In the case of the Australian model, this phenomenon was associated with the lower speed of higher conflicting traffic, enabling shorter accepted gaps. Countering this explanation, the result could simply be a function of shorter headways at high conflicting flows resulting in a lower critical headway, and longer headways at low conflicting flows resulting in a larger critical headway. However, the Maximum Likelihood method used to estimate the critical headway is not sensitive to changes in the traffic flow and hence the average headway size. This relationship has not been investigated further.

The average follow-up headway has a weak correlation to the percentage of heavy vehicles, conflicting flow and geometric parameters. Given that there is no conflicting flow during a follow-up event, the weak correlation (−0.15) of follow-up headway to the conflicting flow is appropriate. In addition, the weak correlation (+0.07) of follow-up headway to the percentage of heavy vehicles appears appropriate.

Isolating the average follow-up headway for single-lane sites yields a moderate positive correlation to the exiting flow (+0.56) and a negative correlation to the width of the splitter island (−0.50) and the diameter (−0.53). As observed in the field, some entering drivers tend to hesitate during an exiting vehicle event. This behavior results in longer follow-up headways. The width of the splitter island is plausibly correlated because it physically separates the entry and exiting movements. The diameter also can be plausibly correlated because of the strong correlation between diameter and the width of the splitter island.

For multilane sites, the right-lane follow-up headway is less likely to be influenced by exiting vehicles because of the larger physical separation. The correlation of the *splitter width plus the left-lane entry width* is much stronger (−0.62). The follow-up headway as a function of the *splitter width plus the left-lane entry width* and the diameter are illustrated in Figures 23 and 24, respectively. The follow-up headway decreases as these geometric parameters increase.

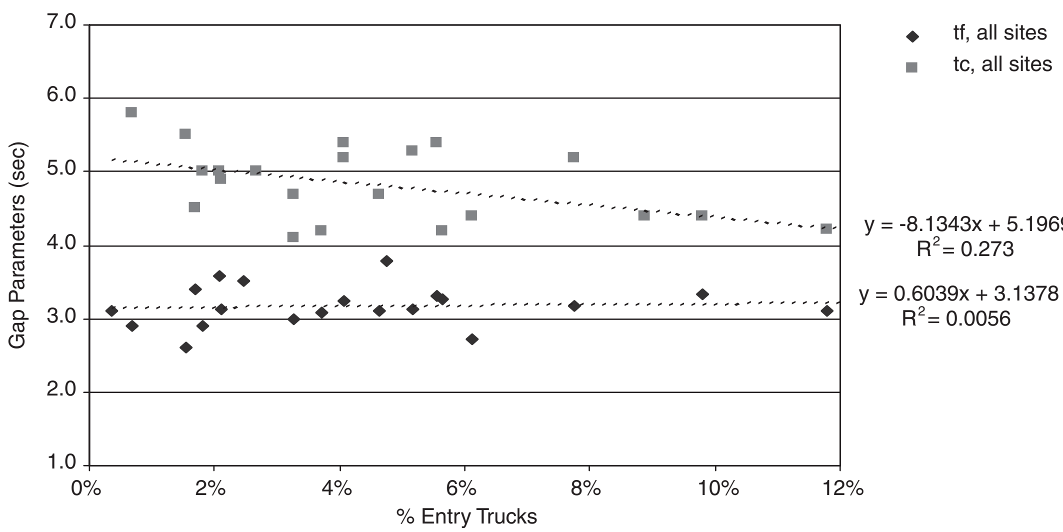

Figure 21. Critical headway and follow-up headway as a function of percentage of entering trucks.

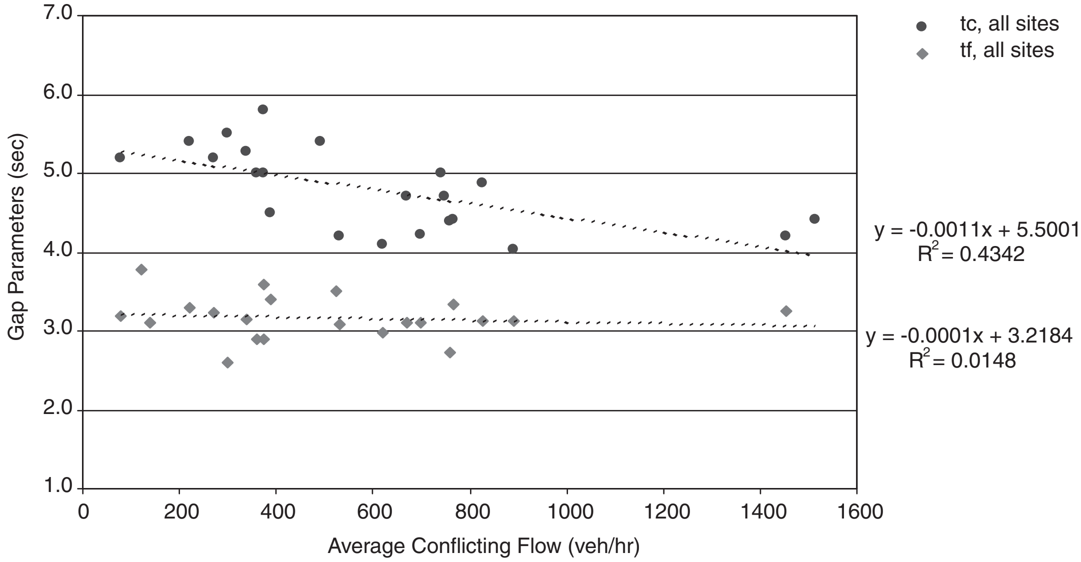

Figure 22. Critical headway and follow-up headway as a function of average conflicting flow.

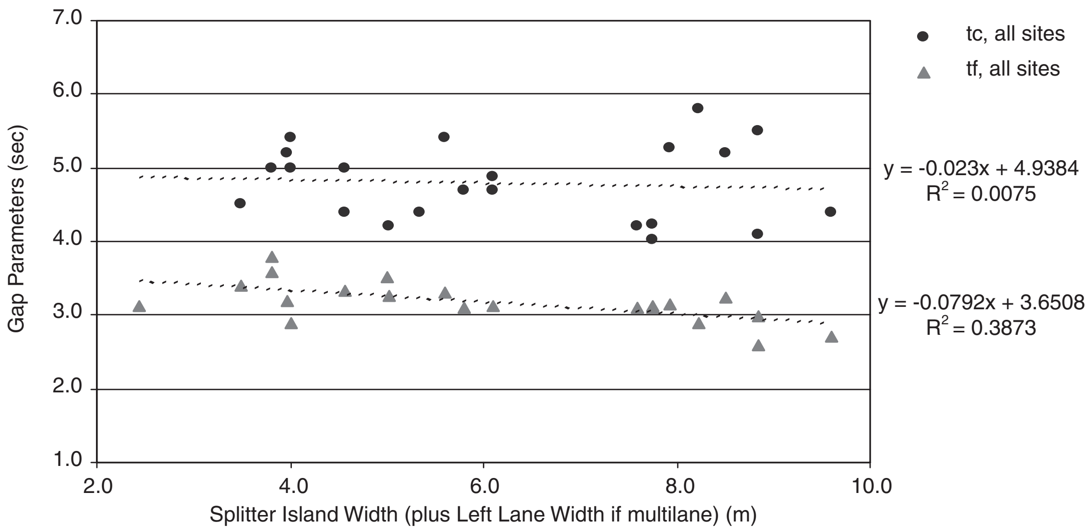

Figure 23. Critical headway and follow-up headway as a function of splitter island width (plus left-lane width if multilane).

The critical headway and follow-up headway as functions of the entry-lane width, entry angle, and radius are illustrated in Figures 25, 26, and 27, respectively. There is a slight and unintuitive increase in the critical headway as the entry-lane width increases. The entry-lane width does not appear to have an influence on the follow-up headway. No strong relationship is indicated between the entry angle or radius and driver behavior.

In each of the correlated relationships, the prediction of the follow-up headway varies between 3.0 and 3.5 s. Based on these estimates, the *y*-intercept of the relationship between entry capacity and conflicting flow would vary between 17 and 20 vehicles/min. The actual data at the *y*-intercept vary between 14 and 25 vehicles/min, and, hence, using the average follow-up headway prediction for all entries would be as effective as using description relationships for the follow-up headway.

The prediction of the critical headway as a function of the conflicting flow varies between 4.0 and 5.2 s. At the single-lane sites with lower average conflicting flows (13 vehicles/min or less), the critical headway varies between 4.7 and 5.2 s. Changes in the critical gap or slope of the entering-conflicting flow relationship result in larger variation to the maximum entering flows at higher conflicting flows. At a conflicting flow rate of 13 vehicles/min, the change in the maximum entering flow is less than 1 vehicle/min.

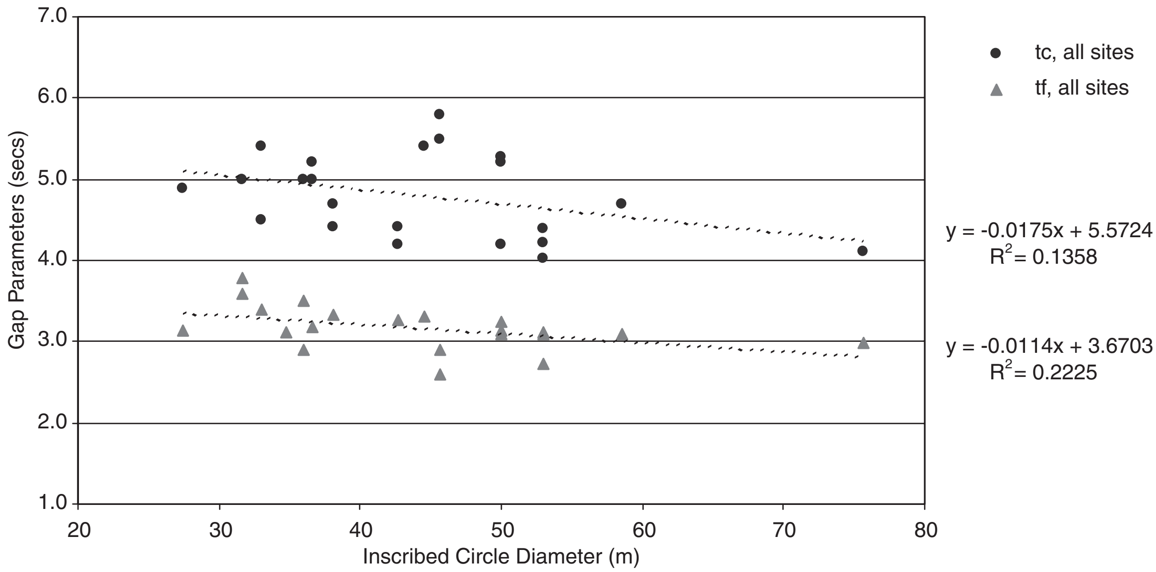

Figure 24. Critical headway and follow-up headway as a function of inscribed circle diameter.

It has also been suggested that as drivers become more familiar with roundabouts, their critical headways and follow-up headways will decrease. Similarly, as drivers are faced with more congested situations, their behavior will become more urgent and the critical headways and follow-up headways will also decrease. Correlation analysis does not suggest any strong relationship between driver behavior and these factors. Although the age of the subject roundabouts varies between 3 and 9 years and the duration of the queue varies between 1 and 30 min, no significant trend can be clearly observed.

Impact of Exiting Vehicles on Driver Behavior

In the critical headway and follow-up headway methodology, the conflicting headway is defined as the headway between two consecutive conflicting vehicles. Because of the presence of an exiting vehicle between conflicting vehicles, some entering vehicles are perceived to reject reasonable gaps or to follow up with hesitation. The impact is a longer estimate of the critical headway and follow-up headway, which is associated with lower field capacity. The effect of exiting vehicles

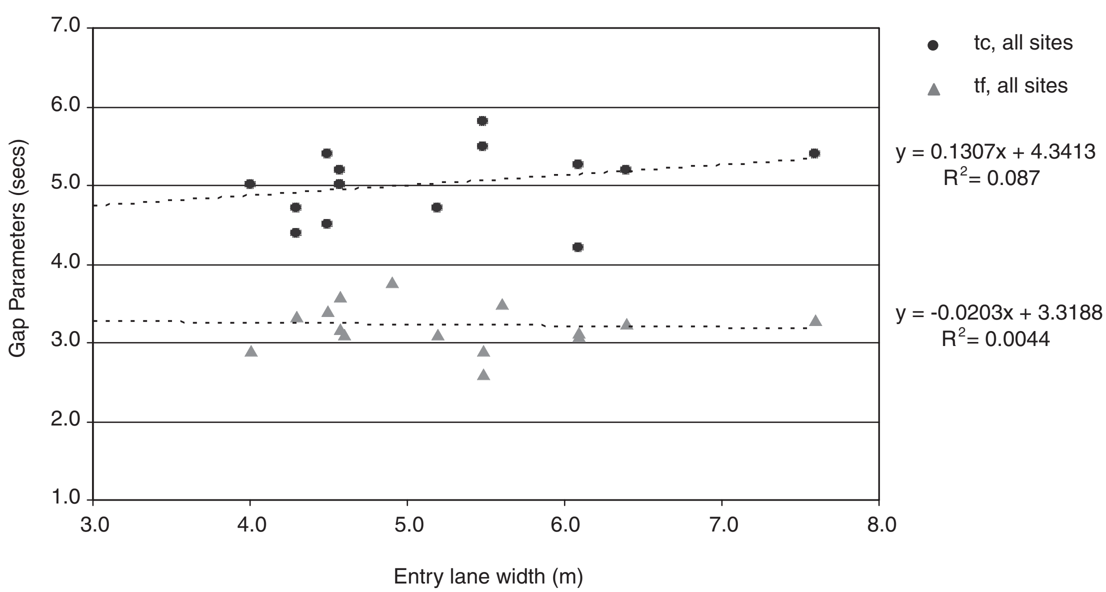

Figure 25. Critical headway and follow-up headway as a function of entry-lane width.

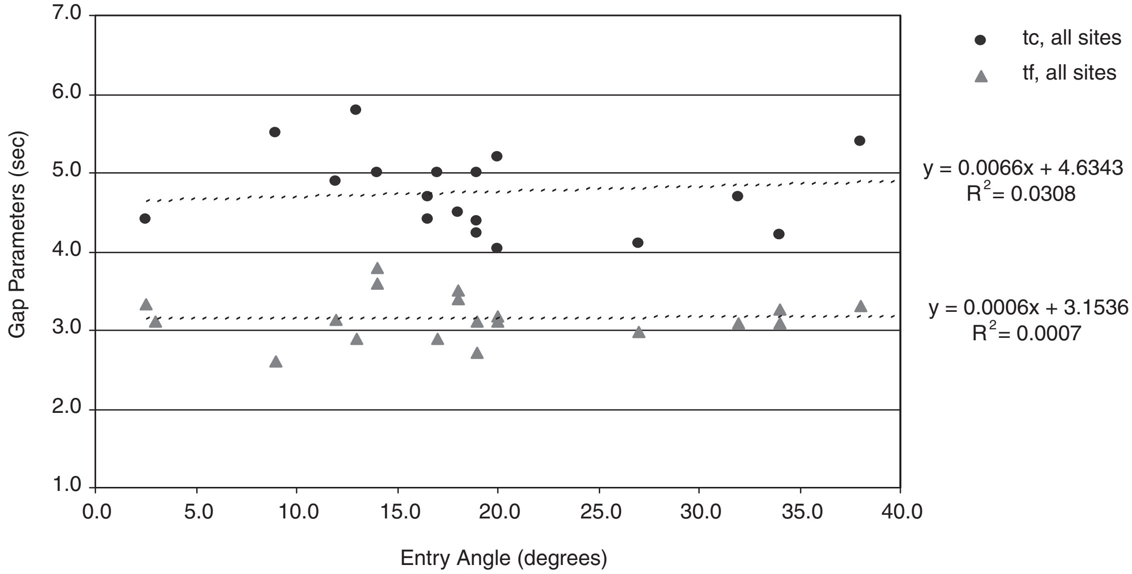

Figure 26. Critical headway and follow-up headway as a function of entry angle.

at single-lane roundabouts has been explored independently by Mereszczak et al. (*32*).

Exiting vehicles may be incorporated in the calculation of the average gap parameters by redefining the conflicting headway when there is an exiting vehicle. One preliminary assumption to simplify the methodology is to include *all exiting vehicles* in the definition of the headway. To do so, the model is adjusted to accommodate the travel time difference between the exit and the conflicting position at the entry line. A vehicle hesitates for an exiting vehicle much the same as it would if the exiting vehicle had been a conflicting vehicle at the entry line. If the first event defining the headway is an exiting vehicle at $t = 10$ s, and the second is a conflicting vehicle at $t = 11$ s, the headway is not $(11-10=)1$ s (which is unrealistic), but rather is calculated as $11-(10+$ the travel time of the exiting vehicle to the conflicting position at the entry line). Travel speed, exit width, and splitter width enable the calculation of the travel time.

The recalculated critical headways for a number of the single-lane sites are provided in Table 43. The critical headways considering all exiting vehicles vary between 3.7 and 4.3 s. These headways are much lower and more consistent than the critical headways assessed without exiting vehicle events, which vary between 4.4 and 5.9 s. In practice, the exiting flow does not

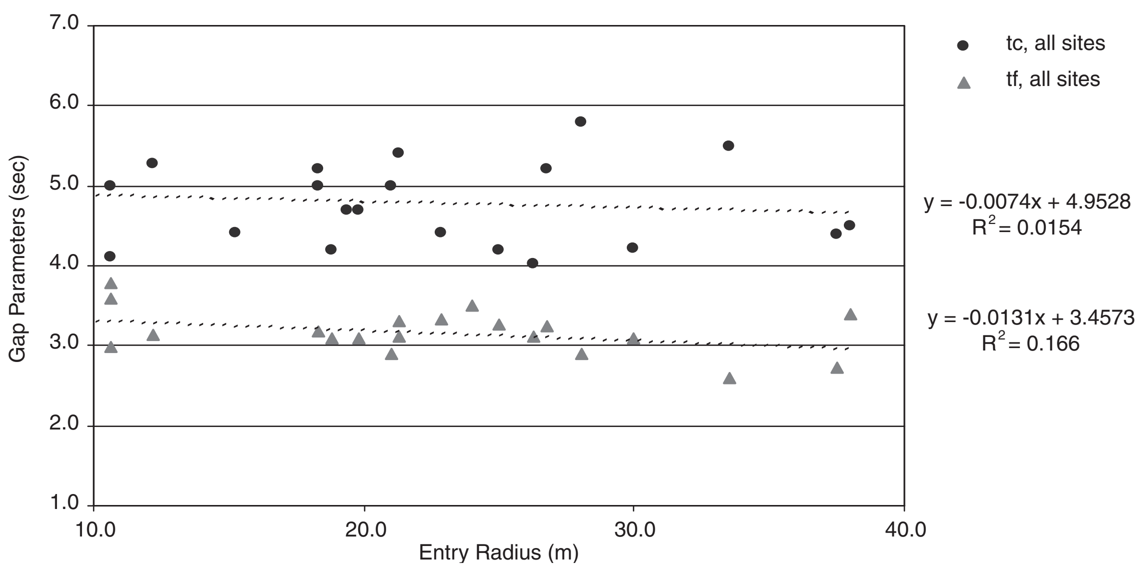

Figure 27. Critical headway and follow-up headway as a function of entry radius.

Table 43. Critical headway adjusted for exiting vehicles.

Site	Critical Headway, 0% Exiting Flow t_c, (s)	Critical Headway, 100% Exiting Flow t_c, (s)
MD06-N	5.2	3.7
MD07-E	5.4	4.1
ME01-E	4.5	4.1
OR01-S	4.7	3.9
WA01-W	4.4	4.0
WA03-S	5.0	3.8
WA04-N	5.2	4.3
WA05-W	5.9	4.2

Table 44. Passenger car equivalency factors.

Model	Trucks	Buses	Motorcycles
UK	NA[1]		
Australian	3	2	1
German	2	1.5	0.5
French	2	2	0.5
Swiss	2	2	0.5
HCM Upper	NA[1]		
HCM Lower			
FHWA	2	1.5	0.5

[1]No equivalency factors specified; FHWA factors used

impact all entering vehicles, and the exact extent of the influence of exiting vehicles has not been determined.

To estimate the capacity, the recalculated critical headway and follow-up headways should be used together with an estimate of the conflicting plus exiting flow. While the gap parameters are shorter, the methodology assumes that the entering vehicle is exposed to more conflicting events. Because the influence of the exiting flow is inherent in the critical headway and follow-up headways that do specifically identify the exiting event, only the conflicting flow is considered.

Capacity Model Calibration

Based on the findings of the evaluation of existing international models, calibration to U.S. conditions appears necessary to improve the quality of capacity estimates. The calibration of the UK, Australian, German, French, Swiss, HCM 2000, and FHWA models was completed in the following step-by-step process:

1. **Passenger Car Equivalents:** The uncalibrated entry capacity has been converted to passenger car units (pcu) using the average percentage of heavy vehicles observed at each entry and the equivalency factors presented for each model in Table 44. For the HCM 2000 and UK models with no recommended equivalency factors, the FHWA factors were used.
2. **Effective Geometry:** A maximum effective entry and approach width of 16 ft (4.8 m) was assumed based on the observed physical use of the lane. The entry width impacts the UK, Australian, French, and Swiss capacity models.
3. **Measured Critical Headway and Follow-Up Headways:** The Australian, German, French, and HCM 2000 models were calibrated using the approach-measured values of critical headway and follow-up headway. While the use of approach-measured gap parameters demonstrates the predictive ability of the models, relationships or the average gap parameters would be used in practice.
4. **Revised *y*-Intercept:** Per the calibration steps recommended by Kimber (*19*), the UK model *y*-intercept constant was calibrated using the field-measured capacity data.

Single-Lane Roundabouts

The RMSE and average error of the single-lane data for each model and calibration step is presented in Tables 45 and 46, respectively. As previously indicated, all uncalibrated models tend to predict much larger entry capacities than observed. The HCM 2000 and German models have lower RMSE than the other models, which is attributable to the long critical headway and follow-up headway. The French and UK models produce the largest error. The French model uses a very short average follow-up headway not observed in the field data. One possible explanation for the error associated with the UK model is that the UK model is based on data collected from sites where wide single-lane entries are uncommon; an entry with a width of 6 m (20 ft) would often be marked as two entry lanes with widths of 3 m (10 ft) each, for example. On the other hand, two of the sites in this database—MD07-E (Taneytown, Maryland) and WA04-N (Port Orchard, Washington)—have large curb-to-curb entry widths but are clearly marked and operated with only a single entry lane. As a result, the UK model effectively treats a wide single-lane entry as having two lanes and therefore overestimates the capacity of a single-lane entry.

There is a slight improvement in the RMSE when the flow inputs are adjusted for heavy vehicles. Because the conflicting flow in passenger car units per hour is larger than the conflicting flow in vehicles per hour, the entry capacity estimates are lower. Furthermore, the measured entry capacity is larger when converted to passenger car units, and hence the difference between measured and predicted entry capacity (the average error) is smaller. Larger equivalency factors could be used to reduce the error further; however, this exercise would not realistically indicate the extent of the influence of heavy vehicles on the entry capacity. Gap parameters already

Table 45. Calibration of single-lane capacity models.

Model	Root Mean Square Error (RMSE)				
	Uncalibrated veh/h	Uncalibrated pcu/h	Calibrated geometry	Calibrated actual t_f & t_c	Calibrated geometry & y-intercept factor F
UK (Kimber/RODEL)	787	773	431	–	164
Australian (Akçelik/SIDRA)	589	545	473	160	
German (Wu/Kreisel)	294	215	–	143	–
French (Girabase)	1163	1138	1147	206	–
Swiss (ETH Lausanne)	373	348	328	–	–
HCM Upper	326	322	–	145	–
HCM Lower	180	187	–	–	–
FHWA (Modified UK)	240	224	–	–	–

include the influence of heavy vehicles and, in examination of these against the percentage of heavy vehicles, did not show an intuitive trend. A more detailed examination of truck factors should be performed outside the model calibration.

There is an improvement in the RMSE when the effective entry and approach geometry is used. As anticipated, there is a significant improvement in the UK model RMSE. The Australian, French, and Swiss models have slight improvement in the RMSE. An effective entry width threshold of 4.6 m (15 ft) was also tested, but it did not result in any additional improvement in the error.

There is a large improvement in the RMSE when the field-measured critical headway and follow-up headway are used. As indicated by the average error, the French model still tends to overpredict the data. The calibrated HCM 2000 and German models yield a slight improvement and have lower RMSE than the other models.

There is a large improvement in the RMSE when the y-intercept of the UK model is calibrated. The UK model has a constant, F, of 303 indicated in the y-intercept of the entry capacity equation (Equation B-6 in Appendix B). Given that the measured entry capacity and conflicting flow is known, the expression for the y-intercept can be rearranged to estimate the local value of this constant. The revised localized intercept for a single-lane entry is reduced by reducing F to 223. A lower constant infers a lower y-intercept and hence

Table 46. Average error for single-lane sites.

Site	n	Australian		UK		German		French		Swiss		HCM Upper		HCM Lower	FHWA
		pcu	Cal.	pcu	Cal.	pcu	Cal.	pcu	Cal.	pcu	Cal.	pcu	Cal.	pcu	pcu
WA08-N	4	+603	+120	+872	+191	+264	+133	+1089	+393	+485	+481	+388	+155	+198	+278
WA08-S	24	+360	−79	+499	−116	−7	−58	+803	+206	+212	+171	+115	−39	−72	+2
MD06-N	14	+317	−158	+270	−105	+112	−20	+882	+164	+134	+133	+267	−18	+146	+93
MD06-S	4	+529	−73	+1187	−45	+149	−39	+1626	+137	+331	+294	+303	−31	+107	+151
MD07-E	56	+398	−74	+410	−7	+190	+39	+952	+221	+241	+238	+337	+44	+206	+178
ME01-E	42	+358	−12	+496	+115	+198	+50	+722	+152	+376	+376	+323	+72	+135	+217
ME01-N	1	—	—	—	—	—	—	—	—	—	—	—	—	—	—
MI01-E	8	+496	+89	+926	+263	+329	+83	+1033	+265	+504	+491	+460	+112	+283	+358
OR01-S	15	+160	−123	+600	+59	+84	−16	+524	+59	+211	+204	+232	+42	+67	+119
WA01-N	3	+131	0	+510	+152	+215	+86	+580	+186	—	—	+365	+148	+206	+254
WA01-W	6	+41	−67	+417	+40	+71	−18	+360	+9	+193	+193	+250	+53	+93	+109
WA03-E	2	—	—	—	—	—	—	—	—	—	—	—	—	—	—
WA03-S	28	+390	−8	+446	+86	+248	+29	+732	+119	+323	+322	+371	+46	+181	+268
WA04-E	15	+505	−66	+602	−120	+102	−54	+1057	+122	+266	+231	+246	−38	+53	+118
WA04-N	85	+572	−100	+784	−47	+120	−48	+1256	+131	+314	+289	+260	−37	+64	+128
WA04-S	4	+307	−53	+800	+100	+110	+58	+882	+124	+278	+249	+242	+99	+65	+140
WA05-W	6	+451	−104	+378	−47	+64	−80	+773	+125	+207	+206	+175	−76	+2	+54
WA07-S	1	—	—	—	—	—	—	—	—	—	—	—	—	—	—

Legend: n = number of observations; pcu = passenger car units; Cal. = calibrated gap parameters and/or calibrated intercept lower entry capacity. As illustrated by the reduction in the RMSE, the impact of this adjustment, in addition to the revised geometry, is significant. The combined average intercept constant of single-lane and multilane roundabouts was also tested. The intercept constant, *F*, decreases from 223 to 205, and the associated error increases.

The capacity models with calibrated data were also tested with larger passenger car equivalency factors. The associated error does not improve. Some of the sites with high truck volumes have positive average errors, and an increase in the passenger car equivalency only serves to increase the error. Similarly, some of the sites with low truck volumes have negative average errors, and an increase in the factors does not have a sufficient impact on the prediction to improve the error.

Average Follow-Up and Critical Headways

Where local field data for the gap parameters cannot be readily obtained, the user will likely rely on the availability of the national data collected as part of this study. Two sets of constants were tested: the measured weighted average t_c and t_f (5.1 s and 3.2 s, respectively), and the t_c and t_f required to minimize the HCM 2000 model's RMSE (5.4 s and 3.2 s, respectively). The gap parameters required to minimize the HCM 2000 model's RMSE are similar to the average weighted field parameters.

Multilane Roundabouts

The RMSE and average error for the calibration of various models to the multilane data are presented in Tables 47 and 48, respectively. For the approach-based models, the entry capacity data were only used if found plausible in both the left and right lanes.

When adjusted for heavy vehicles, most models show a slight improvement in ability to predict measured entry capacity. However, the RMSE for the HCM 2000 model does not improve. The uncalibrated HCM 2000 model both under- and overpredicts the data, and any increase in the conflicting flow generally reduces the predicted entry capacity.

Table 47. Calibration of multilane capacity models.

Model	Root Mean Square Error (RMSE)				
	Uncalibrated veh/h	**Uncalibrated pcu/h**	**Calibrated geometry & *y*-intercept *F***	**Calibrated *F* (all sites)**	**Calibrated actual t_f & t_c (move-up time < 6 s)**
UK (Kimber/RODEL)	1054	982	270	324	—
Australian (Akçelik/SIDRA) • Right lane • Left lane	 473 459	 476 473	—	—	 161 190
German (Wu/Kreisel)	**375**	**307**	—	—	252
French (Girabase)	818	692	—	—	230
Swiss (ETH Lausanne)	490	392	—	—	—
HCM 2000 • Upper • Lower	 **271** 367	 **320** 426	—	—	372
FHWA (Modified UK)	953	857	—	—	—

Table 48. Average error for multilane sites.

Site	*n*	Australian right lane		Australian left lane		UK		German		French		Swiss	HCM Upper		HCM Lower	FHWA
		pcu	Cal.	pcu	Cal.	pcu	Cal.	pcu	Cal.	pcu	Cal.	pcu/ Cal.	pcu	Cal.	pcu	pcu
WA09-E	194	+479	–109	+521	+145	+905	+71	–45	–104	–1	–310	+32	–502	–451	–379	+609
MD04-E	36	+437	+195	+401	+171	+884	+89	+431	+409	+648	+119	+617	–343	–256	–181	—
MD05-NW	31	+173	+28	—	—	+205	+229	+37	+100	+519	+223	+237	+24	+185	+137	—
MD05-W	3	+264	+24	—	—	—	—	+264	+214	+545	+199	+357	–53	–9	+53	—
VT03-E	16	+397	–180	+411	–70	+1131	+263	+331	–94	+687	–181	+478	–309	–453	–163	+930
VT03-S	20	+448	–209	+631	+186	+1748	+193	+182	+103	+1034	–62	+250	–562	–506	–404	+716
VT03-W	83	+367	–66	+491	+133	+1054	+179	+283	+263	+631	+222	+386	–429	–341	–275	+829

Legend: *n* = number of observations; pcu = passenger car units; Cal. = calibrated gap parameters and/or calibrated intercept

It should be noted that the HCM 2000 model is not intended to predict capacity of a multilane entry.

There is a moderate to large improvement in the RMSE when the field-measured critical headway and follow-up headway are used. The Australian model uses an approach-based critical headway and a lane-based follow-up headway. The German, French, and HCM 2000 models use an approach-based critical headway and follow-up headway. As the field-measured critical headway and follow-up headway were determined by lane, the right-lane parameters were assumed to apply to the approach. There is a large improvement in the Australian capacity estimates, as the field gap parameters are longer than published parameters (as noted previously). There is very little change in the calibrated German model, mostly because the estimate of the critical headway does not differ significantly from the published parameters. The French model assumes a very short follow-up headway that was not observed in the U.S. data; an increase in the follow-up headway greatly improved the estimates. The calibrated HCM 2000 model capacity estimates lie between the HCM 2000 upper-bound and lower-bound models (with a little more variation than captured by the HCM 2000 model limits). It should be noted that the lower-bound model results represent MD05-NW (Towson, Maryland), which has a single-lane entry and as such has approximately half the capacity of the multilane entries.

The y-intercept constant, F, in the UK model was recalibrated to local conditions and reduced from 303 to 220. While the approach error typically improves, the capacity estimate for MD05-NW is low. The recalibration of the y-intercept overcompensates the required adjustment needed at this site.

Incorporating the Effects of Exiting Flow

Because of the presence of an exiting vehicle, some entering vehicles were perceived to reject reasonable gaps or to follow-up with hesitation. The impact is a longer estimate of the critical headway and follow-up headway, which is associated with lower field capacity. Assuming that an exiting vehicle always impacts the entering driver behavior, the gap parameters were recalculated for single-lane sites. In practice, the exiting flow does not impact all entering vehicles; however, the exact extent of the influence of exiting vehicles was not determined in this exercise.

To estimate the capacity, the recalculated critical headway and follow-up headways were used with an estimate of the "conflicting plus exiting flow," rather than just the conflicting flow. While the gap parameters are shorter, the methodology assumes that the entering vehicle is exposed to more conflicting events.

The HCM 2000 model using the revised critical headway and follow-up headways is illustrated in Figure 28. For any given conflicting flow, the variation in the capacity estimate is due to the inclusion of exiting flow. The estimated variation in the capacity appears to be similar to the spread in the field data. However, there are two issues: (1) the RMSE is much higher with the inclusion of the exiting flow, and (2) the prediction of high conflicting flow is poor. The increase in the RMSE also occurs in the Australian and German capacity estimates.

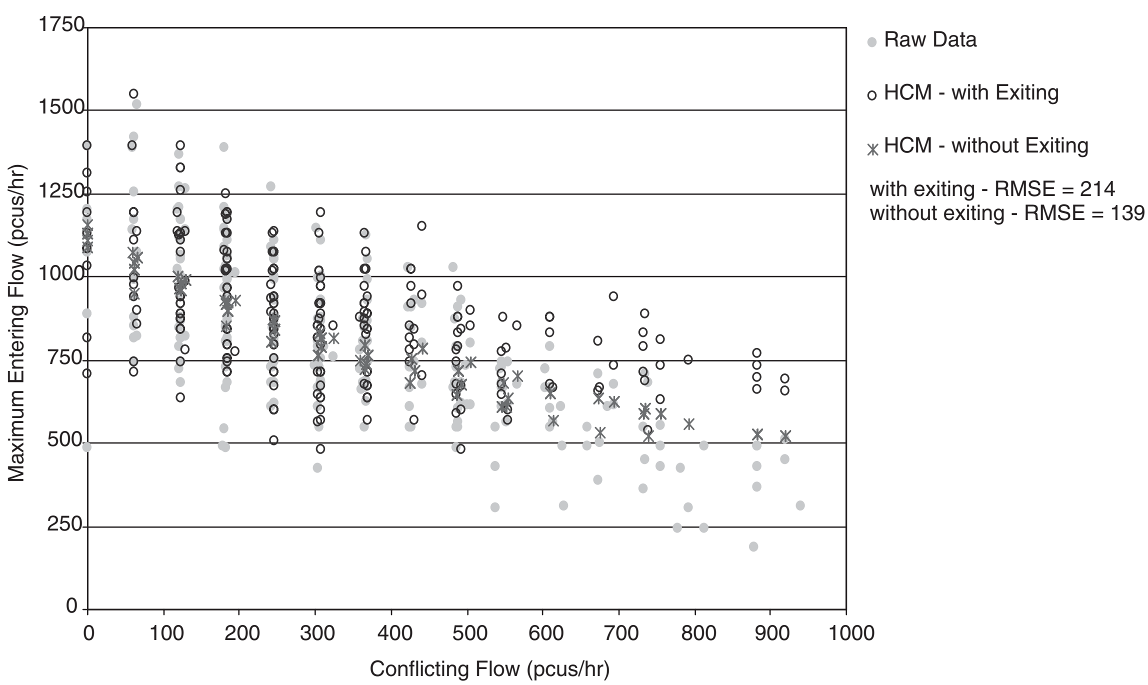

Figure 28. Effect of exiting vehicles on estimated capacity.

In summary, the inclusion of exiting vehicles in the analysis methodology did not improve the estimate of the capacity. However, because such behavior was observed in the field, refinements to the assumptions may suggest otherwise. Where the exiting event was not incorporated in the methodology, the derived gap parameters inherently include the influence of the exiting vehicles. That is, any hesitation due to an exiting vehicle is indicated as a long follow-up headway or rejected critical headway. While this approach does not predict as much variation as seen in the field data, it does more accurately reflect the average condition.

Capacity Model Development

Based on the analysis, this study determined that several models—ranging in complexity from simple regression models to more complex analytical (e.g., SIDRA) and regression (e.g., UK) models—can be recalibrated to achieve essentially the same goodness of fit. This section compares the goodness of fit of a simple regression model against other model forms presented previously, followed by the recommended model form.

Single-Lane Capacity Model Development

Regression curves (both linear and exponential) are illustrated in Figure 29, along with the class mean of the data (the mean observed capacity value for each conflicting flow value). Both regressions constitute a good representation of the class means of the entry capacity. The coefficient of determination (R^2) of the linear and exponential relationships is approximately 0.5.

Figure 30 illustrates a plot of two capacity estimates: the capacity estimate using the HCM 2000 model and average field values for the gap parameters, and the capacity estimate using exponential regression of the data. The exponential regression model yields a RMSE of 155, which is slightly better than the RMSE of 160 from the HCM 2000 model. The error also compares favorably to the error predicted by the calibrated models presented previously, with the lowest RMSE of 145, obtained using the approach gap parameters within the HCM model form. Both models tend to overestimate capacities at higher circulating flows.

Upon closer inspection, the form of the HCM 2000 model can be transformed to be similar to that of the regression model. The HCM 2000 model form is as follows:

$$q_{e,\max} = \frac{q_c \exp(-q_c t_c/3600)}{1-\exp(-q_c t_f/3600)} \tag{4-1}$$

where

$q_{e,max}$ = entry capacity (veh/h)
q_c = conflicting circulating traffic (veh/h)
t_c = critical headway (s)
t_f = follow-up headway (s)

The HCM 2000 expression can be simplified to yield the following:

$$q_{e,\max} = \frac{3600}{t_f}\exp\left(-\frac{t_c - t_f/2}{3600}q_c\right) \tag{4-2}$$

which is of the same form as

$$q_{e,\max} = A\cdot\exp(-B\cdot q_c) \tag{4-3}$$

where

$A = 3600/t_f$
$B = (t_c - t_f/2)/3600$
t_c = critical headway (s)
t_f = follow-up headway (s)

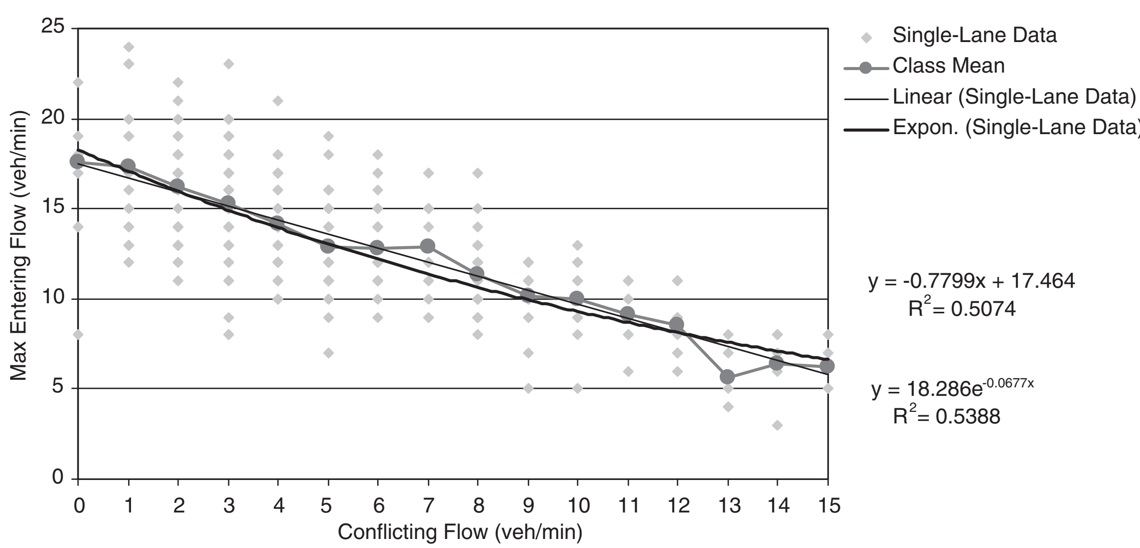

Figure 29. Linear and exponential regression and class means prediction for the single-lane entry capacity.

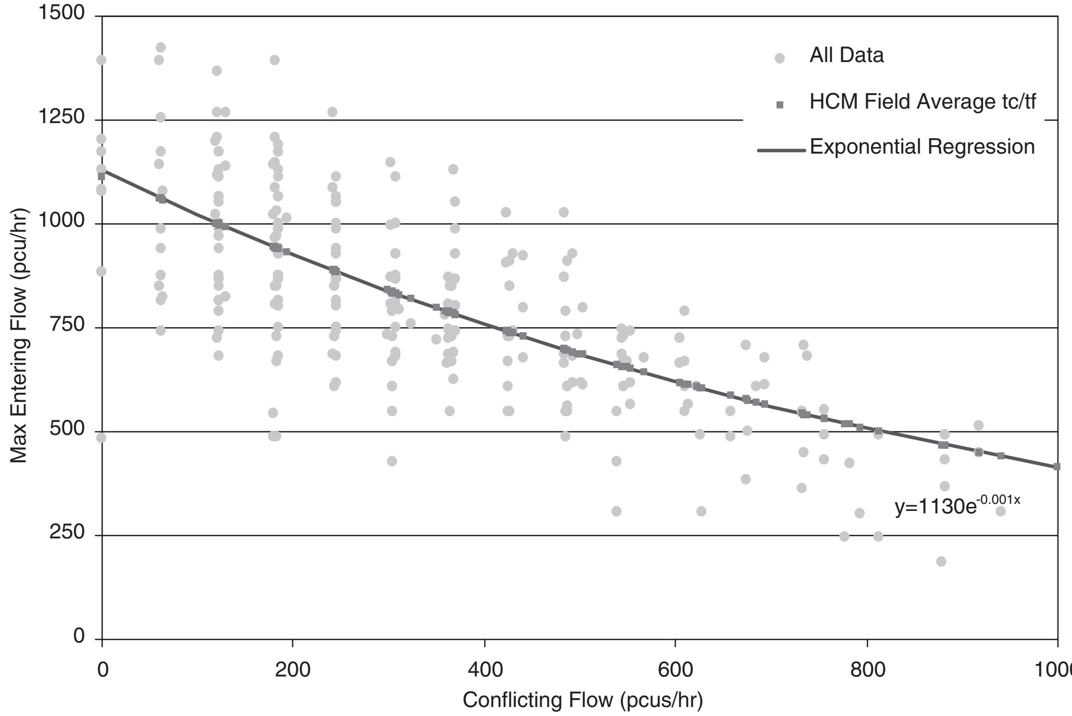

Figure 30. Capacity using HCM and exponential regression models.

The predicted exponential regression intercept and slope of 1129 and 0.0010 compares favorably with the HCM intercept of ($3600/t_f = 3600/3.2 =$)1125 and slope of [($t_c - t_f/2$)/3600= (5.1−3.2/2)/3600=]0.0010. Such findings expand the practical application of the exponential regression and allow users in the future to calibrate the constants against local data.

Based on the above findings, the following form (using variables consistent with current HCM practice) is recommended for the entry capacity at single-lane roundabouts:

$$c = 1130 \cdot \exp(-0.0010 \cdot v_c) \quad (4\text{-}4)$$

where

$c = q_{e,max}$ = entry capacity (veh/h)

$v_c = q_c$ = conflicting circulating traffic (pcu/h).

The model parameters can be calibrated using local values of the gap parameters, per Equation 4-3.

Multilane Capacity Model Development

The multilane data were extracted during periods of queuing in the left *or* the right lane. Because some data for a given lane do not represent a queued condition, a plausibility check of the data was performed. The plausibility of the right- and left-lane entry capacity is given by the following equation:

$$20 - q_c \le q_{e,\max} \le 50 - q_c \quad (4\text{-}5)$$

where

$q_{e,max}$ = entry capacity (veh/h)

q_c = circulating flow (veh/h).

Using these thresholds, the right- and left-lane entry capacity data are reduced from 400 to 385 and 121 observations, respectively. The right-lane data also include observations at MD05-NW (Towson, Maryland), which is a single-lane entry.

The multilane linear regression, exponential regression, and class mean of the entry capacity data for the right and left lanes are illustrated in Figures 31 and 32, respectively. At high conflicting flows, the right-lane linear regression tends toward zero entry capacity. The exponential model is a better fit and a good representation of the class means of the entry capacity. The coefficient of determination, R^2, of the linear and exponential relationships for the right lane is 0.49 and 0.57, respectively. The regression constants are very similar to those predicted for the single-lane data.

Despite the plausibility check, the left-lane data are lower than the right-lane data. The data from MD04-E (Baltimore County, Maryland), which has a dominant left-entry flow, more reasonably reflects the observed entry capacity in the right lane. Beyond a conflicting flow of 20 vehicles/min, data are limited, as reflected by the variation in the class mean. The coefficients of determination for the linear and exponential relationships for the left lane are 0.31 and 0.33, respectively.

The true measurement of entry capacity requires the presence of a queue in both lanes. Therefore, to predict entry capacities, plausible data must be available in both the left and right lanes, which reduces the available data from 400 to 110 observations. The multilane linear and exponential regression and the class mean for the entry are illustrated in Figure 33. There are limited data at high conflicting flows, as

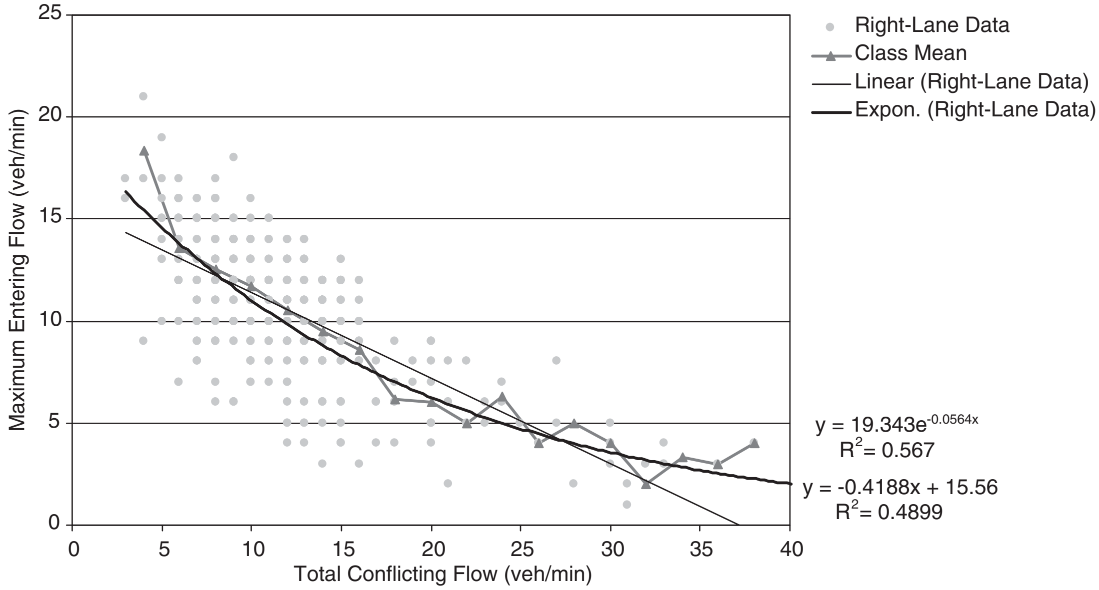

Figure 31. Regression and class mean prediction for capacity of right lane.

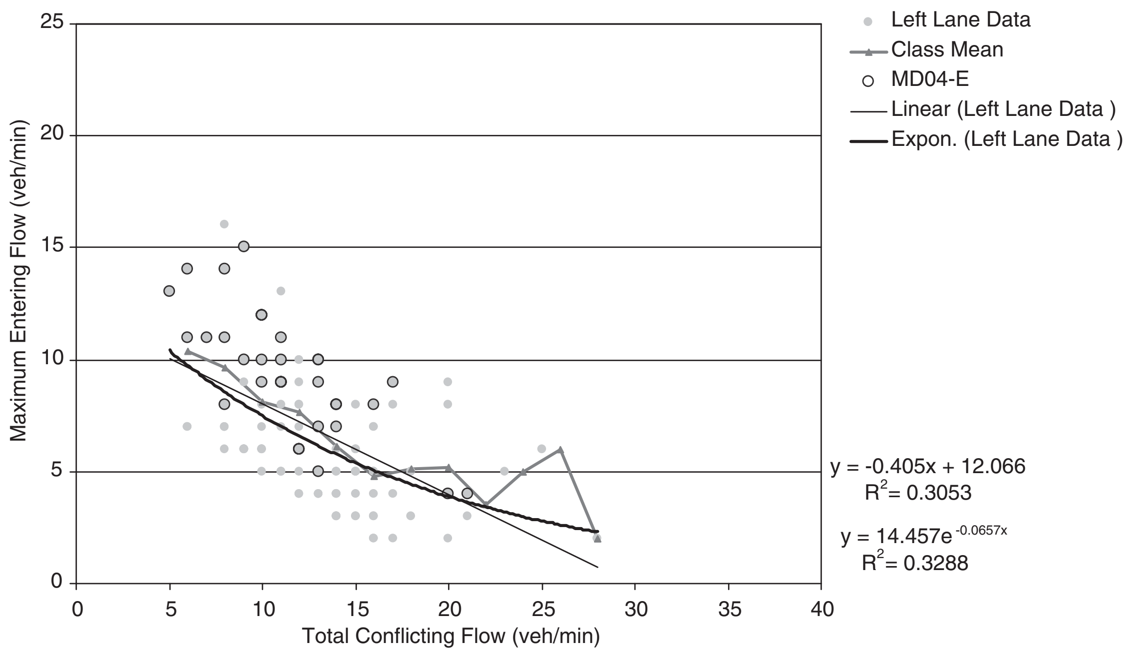

Figure 32. Regression and class means prediction for capacity of left lane.

reflected by the variation in the class mean. The coefficients of determination of the linear and exponential relationships for the left lane are 0.50 and 0.53, respectively. The regression predictions are a reasonable representation of the class mean at lower conflicting flows.

One of the challenges in developing a regression model for a multilane entry is incorporating the flexibility to accommodate different lane configurations and the variety of factors that may influence the utilization of those lanes such as lane designation (e.g., left turn only versus left through), turning movement patterns, downstream influences on lane choice, and driver discomfort with the inside lane. Currently, too few sites in the United States are operating at capacity to allow the development of a separate regression model for each entry and circulating configuration; however, a lane-based model can be developed that allows the consideration of many of these factors.

To maximize the available plausible data, a regression model based on the maximum entry volume in the *left or right lane* has been developed. In short, this model represents the capacity

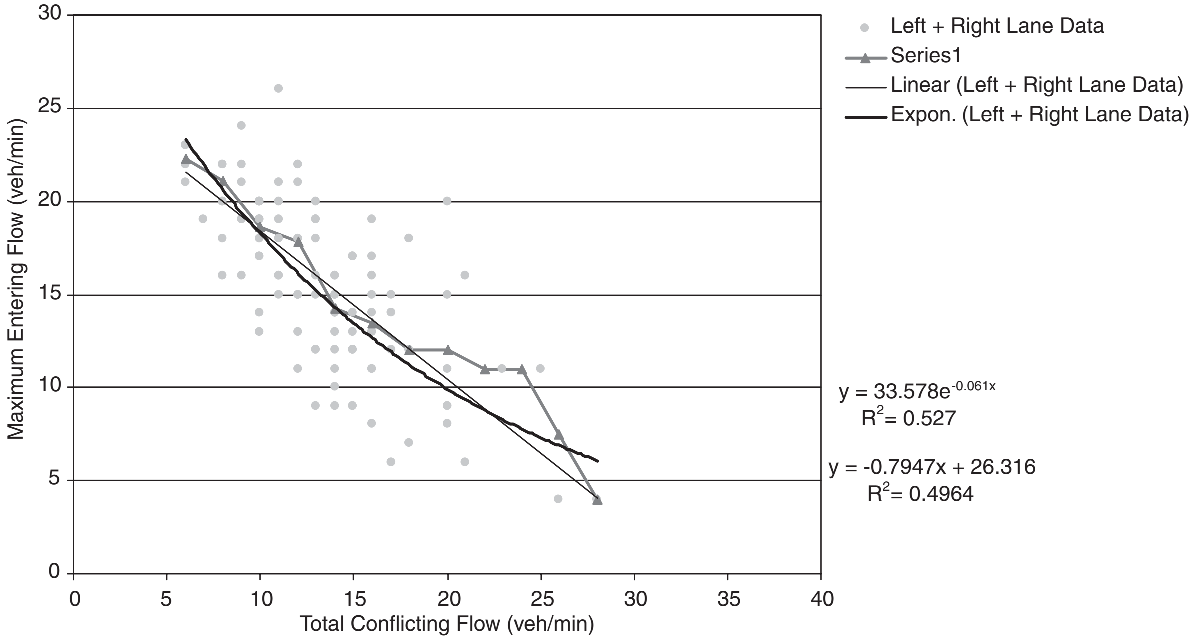

Figure 33. Regression and class means prediction for entry capacity.

of the most heavily utilized lane, or critical lane, thus removing the number of entering lanes from the capacity model. Effectively, a capacity condition is reached when the critical lane has a constant queue. This method is akin to the concepts related to signalized intersections, where one of the lanes is assumed to have a higher flow rate than the others.

The critical-lane data are illustrated in Figure 34. The RMSE of the regression is 145, which is better than the multilane model's RMSE (170 to 250) demonstrated previously.

The linear regression tends toward zero at a conflicting flow of 2,300 pcu/h. The exponential regression provides a better fit and has a higher coefficient of determination. The exponential regression is given by the following equation:

$$q_{e,\max,crit} = 1230 \cdot \exp(-0.0008 \cdot q_c) \qquad (4\text{-}6)$$

where

$q_{e,max,crit}$ = capacity of the critical lane (pcu/h)
q_c = conflicting flow (pcu/h)

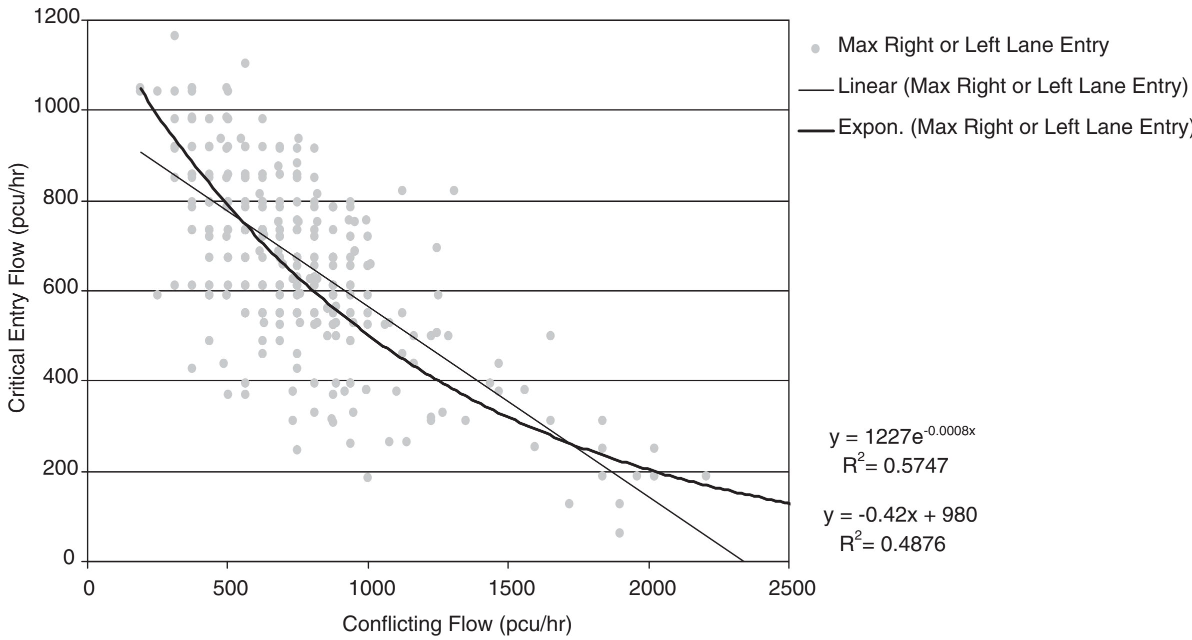

Figure 34. Critical-lane entry data.

The following influences in the multilane data should be noted:

- The critical-lane data mostly comprise right-entry-lane observations. The critical right- and left-lane observations are illustrated in Figure 35. Regression suggests that the critical left-lane capacity is lower; however, there are limited left-lane observations, most of which occurred at MD04-E (Baltimore County, Maryland). Because of limited critical left-lane observations, it is difficult to establish if there is any difference in the capacity between the right lane and left lane and therefore there is insufficient evidence to suggest the need for factors that correct the regression.
- The critical-lane data mostly comprise behavior of entering vehicles against two conflicting lanes. MD04-E has only one conflicting lane, but too few observations to draw any conclusions.

The single-lane and multilane critical-lane exponential regressions are illustrated together in Figure 36. The single-lane regression has a lower intercept than the multilane critical-lane regression. This finding is not intuitive because the average follow-up headways for the single-lane and multilane sites were found to be approximately the same; consequently, the intercepts should be the same. The disparity in the intercept may be caused by a number of reasons, including the lack of multilane critical-lane data observations at the intercept.

The slopes of the single-lane and multilane regressions are also different. Again, the lack of multilane critical-lane data at the intercept and observations at higher conflicting flows may explain this result. Furthermore, if right-lane entering vehicles can accept a gap alongside a left-lane conflicting vehicle, then the actual conflicting flow may be slightly lower than the "total" conflicting flow. This event may be more likely for low conflicting flow conditions, which would influence the regression slope.

Fixing the critical-lane intercept to the single-lane intercept of 1130, and adjusting the slope to minimize the error, yields a RMSE of 145. The single-lane and adjusted multilane critical-lane model is shown in Figure 37.

Based on these findings, the recommended capacity model for the critical lane of a multilane (two-lane) roundabout is as follows:

$$c_{crit} = 1130 \cdot \exp(-0.0007 \cdot v_c) \qquad (4\text{-}7)$$

where

$c_{crit} = q_{e,max,crit}$ = capacity of the critical lane (pcu/h)

$v_c = q_c$ = conflicting flow (pcu/h)

The multilane critical-lane regression model also can be calibrated using the parameters presented previously in Equation 4-3. This finding allows for the development of localized constants for the critical-lane regression model, thereby extending its use to accommodate changes in driver behavior.

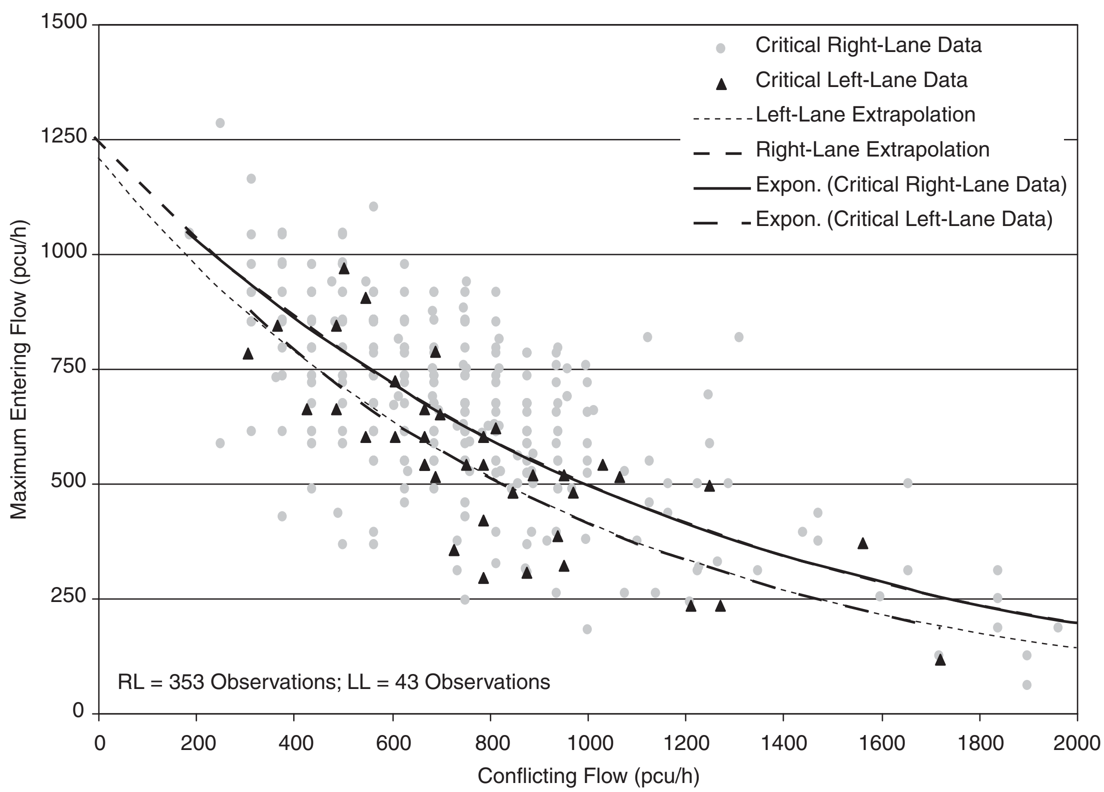

Figure 35. Critical right-lane and left-lane entry data.

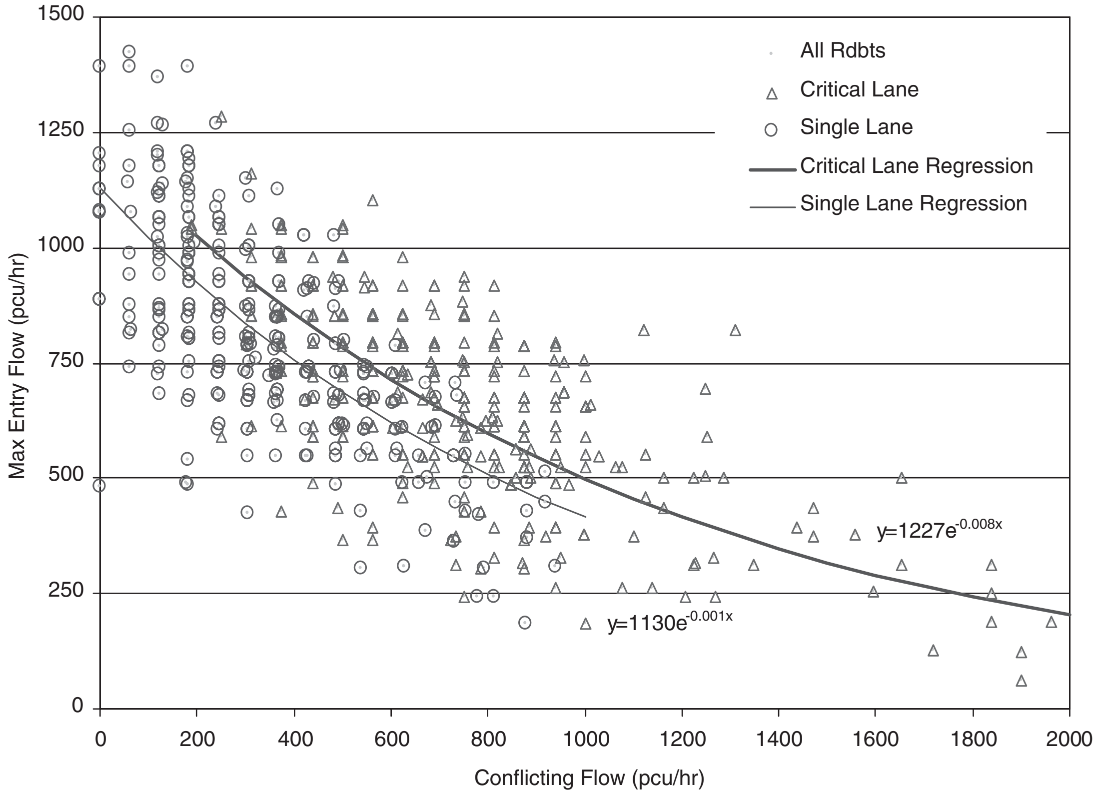

Figure 36. Single-lane and multilane critical-lane regression.

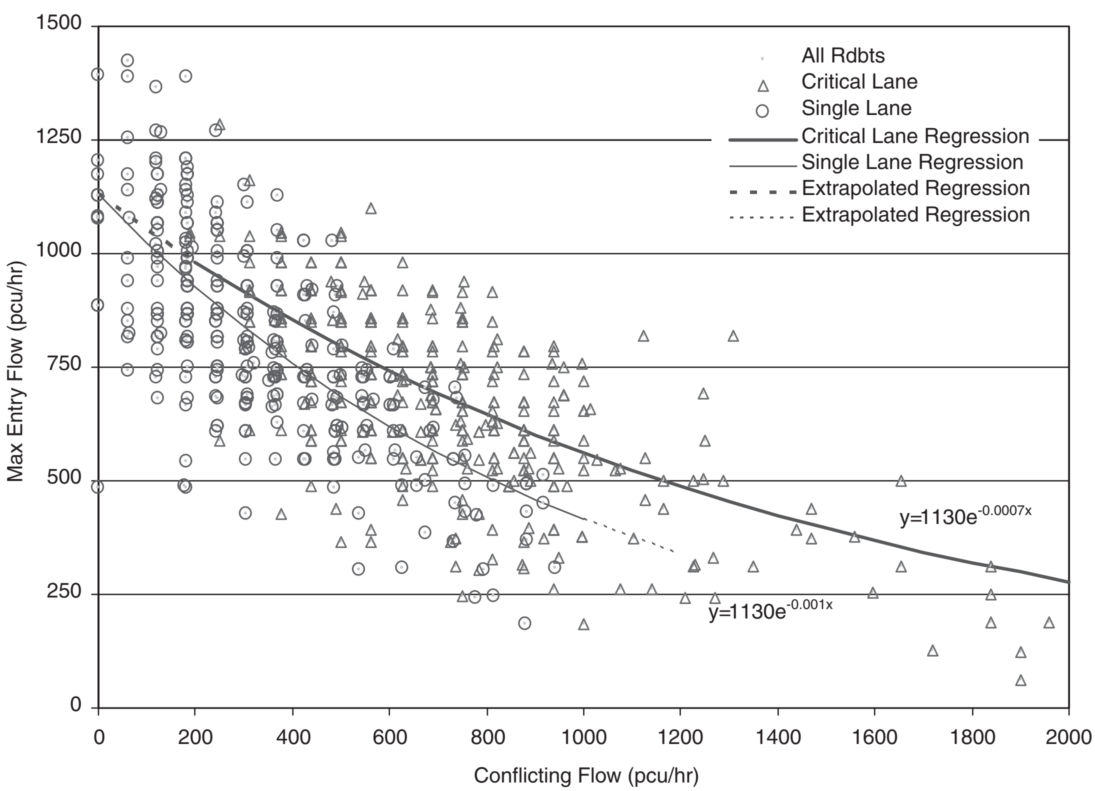

Figure 37. Single-lane and adjusted multilane critical-lane regression.

Level of Service

In the HCM 2000 (*2*), the criteria for levels of service for stop-controlled intersections and signalized intersections differ because the intersection types create different user perceptions. A signalized intersection is expected to carry higher traffic volumes and have longer delays. Also, with the exception of permissive left turns, drivers are protected from other movements during their go phase and are not expected to find their own gap. To determine the intersection LOS at a signalized intersection, the average control delay for the entire intersection is commonly used. On the other hand, the delay and the LOS for two-way-stop–controlled intersections are calculated for only the minor streets because through traffic on the major street is generally not impeded. The difference in how critical or average delay is calculated, coupled with different LOS thresholds, makes comparing the different intersection types difficult.

Using a procedure similar to that used by drivers at two-way-stop–controlled intersections, drivers at roundabouts are required to find their own gap. The fundamental difference is that all drivers entering the roundabout are required to yield to conflicting traffic. In addition, drivers performing left-turn movements are required to find a gap in only the one direction of travel. While these differences may warrant new delay thresholds for roundabouts, the magnitude of the roundabout delay data generally supports using the unsignalized intersection thresholds in HCM 2000.

The proposed LOS criteria for roundabouts are given in Table 49 and are the same as the LOS criteria for stop-controlled intersections in HCM 2000. The LOS for a roundabout is determined by the computed or measured control delay for each lane. Defining the LOS for the intersection as a whole is not recommended because doing so may mask an entry that is operating with much higher delay than the others.

Table 49. Proposed LOS thresholds for roundabouts.

Level of Service	Average Control Delay (s/veh)
A	0 – 10
B	> 10 – 15
C	> 15 – 25
D	> 25 – 35
E	> 35 – 50
F	> 50

Conclusion

The operational analysis described in this chapter results in a new set of proposed capacity models for single-lane roundabouts and for the critical lane of two-lane roundabouts; these models fit better than any existing models, even with calibration. The effect of geometry appears to be most pronounced in terms of number of lanes (a first-order effect); the effect of fine-tuned geometric adjustments (e.g., lane width, diameter) does not appear to have as significant an effect. The current equations used for calculating control delay appear to be reasonable for use at roundabouts, with a possible adjustment at low volume-to-capacity ratios. Levels of service have been defined to correspond with other unsignalized intersections. Further discussion of the significance and applicability of these findings can be found in Chapter 6.

CHAPTER 5

Design Findings

This chapter presents the design findings for this project. The following sections discuss an analysis of predicted versus observed speeds, pedestrian behavior, and bicyclist behavior. The chapter concludes with additional design findings drawn from the safety and operational analyses presented in previous chapters.

Speed Analysis

The speed of vehicles through a roundabout is widely considered to be one of the most important parameters in designing a roundabout. Therefore, the ability to predict the speeds that vehicles will take when traveling through a proposed design is fundamental. To address this need, the research team conducted a detailed analysis of predicted speeds versus actual field-measured speeds to determine how well existing techniques predict reality.

Upon predicting values of entry speed (V_{1p}), through-movement circulating speed (V_{2p}), through-movement exit speed (V_{3p}), and left-turn-movement circulating speed (V_{4p}) and recording actual speeds, V_a, at the same locations, the research team compared the predicted and actual values for V_1 through V_4. The datasets for the single-lane and multilane sites were evaluated separately in order to compare similar geometry. Definitions for each speed variable are provided in Appendix G.

The following sections highlight the findings for each speed value evaluated as part of this report. They are organized in the following order to reflect an increasing degree of uncertainty with each parameter:

- Reproduction of FHWA speed prediction
- Left-turn-movement circulating speed
- Through-movement circulating speed
- Exit speed
- Entry speed

Reproduction of FHWA Speed Prediction

The speed prediction formula presented in FHWA's *Roundabouts: An Informational Guide* (*1*) is based on the basic highway design principles found in the AASHTO's *A Policy on Geometric Design of Streets and Highways* (*33*). The basic relationship among speed, vehicle path radius, superelevation, and side friction factor is as follows:

$$V = \sqrt{15R(e+f)} \qquad \text{(5-1a, U.S. Customary)}$$

where
V = speed (mph)
R = vehicle path radius (ft)
e = superelevation (ft/ft)
f = side friction factor

$$V = \sqrt{127R(e+f)} \qquad \text{(5-1b, Metric)}$$

where
V = speed (km/h)
R = vehicle path radius (m)
e = superelevation (m/m)
f = side friction factor

The FHWA Roundabout Guide presents its speed methodology using a series of graphs to demonstrate the relationship among these parameters, recognizing that side friction factor varies with speed. This process can be simplified by fitting an equation to the relationship between speed and path radius for the two most common superelevation values, $e = +0.02$ and $e = -0.02$. These fitted equations (with a coefficient of determination exceeding 0.997) are as follows:

$$V = 3.4415R^{0.3861}, \text{ for } e = +0.02 \qquad \text{(5-2a)}$$

$$V = 3.4614R^{0.3673}, \text{ for } e = -0.02 \qquad \text{(5-2b)}$$

where

V = predicted speed (mph)

R = radius of vehicle path (ft)

$$V = 8.7602 R^{0.3861}, \text{ for } e = +0.02 \qquad (5\text{-}3a)$$

$$V = 8.6164 R^{0.3673}, \text{ for } e = -0.02 \qquad (5\text{-}3b)$$

where

V = predicted speed (km/h)

R = radius of vehicle path (m)

The original FHWA graphs and the associated fitted equations are shown in Figures 38 and 39 for U.S. customary units and metric units, respectively.

Left-Turn-Movement Circulating Speed

Table 50 summarizes the characteristics of the V_4 data used for this analysis. As the table shows, 1,007 of the 1,231 observations, or 82%, occur at sites with at least 15 observations; only these observations were used in determining percentiles.

Three percentiles—average (mean), 85th-percentile, and 95th-percentile—were examined to determine the best fit to predicted values. As shown in Table 51, the 85th-percentile values for each site with 15 or more observations result in the lowest mean deviation and lowest RMSE of the three percentiles considered.

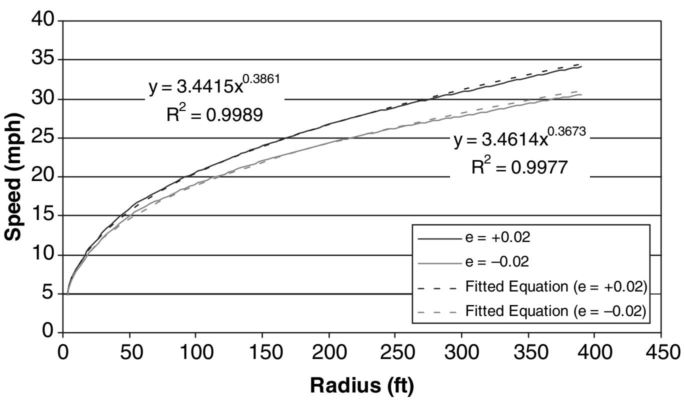

Figure 38. Fitted equation for FHWA speed-radius curves (U.S. customary).

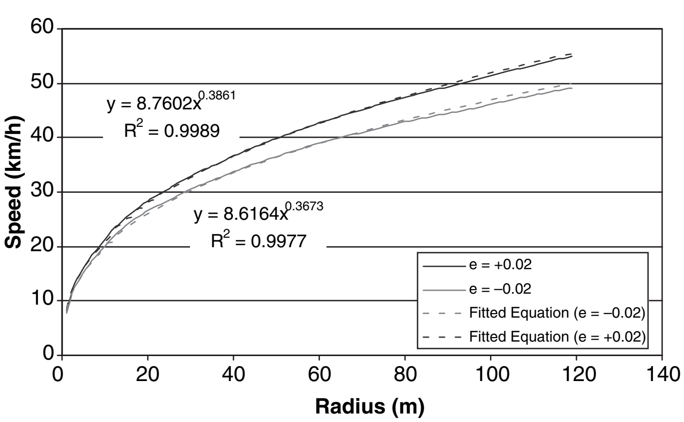

Figure 39. Fitted equation for FHWA speed-radius curves (metric).

Table 50. Summary of left-turn-movement circulating speed (V_4) data.

Characteristic	Total
Range of measured speeds	7 – 28 mph (11 – 45 km/h)
Total number of sites	69
Total number of observations	1,231
Number of sites with 15+ observations	43
Number of observations at sites with 15+ observations	1,007

Figures 40, 41, and 42 present plots of predicted versus actual speeds for all sites, the subset of single-lane sites, and the subset of multilane sites, respectively, along with 85th-percentile values for all sites with 15 or more observations. As can be seen from the figures, the current V_4 method predicts 85th-percentile speeds remarkably well.

Based on these findings, the remaining analysis presented in this report assumes the following:

- The factors influencing the relationship between path radius and speed—side friction factor and superelevation—are reasonable, and no adjustments to side friction factors appear to be necessary.
- The V_4 method is a reasonable predictor of 85th-percentile speed; thus, all subsequent analyses will use 85th-percentile speeds.

Through-Movement Circulating Speed

Table 52 summarizes the characteristics of the V_2 data. As the table shows, 756 of the 990 observations, or 76%, occur at sites with 15 or more observations. These sites with 15 or more observations were used in determining percentiles.

Figures 43, 44, and 45 present plots of predicted versus actual speeds for all sites, the subset of single-lane sites, and the subset of multilane sites, respectively, along with 85th-percentile values for all sites with 15 or more observations. Table 53 summarizes the goodness-of-fit analysis. As can be seen from Figure 43, the current method for predicting V_2 speeds generally overestimates 85th-percentile speeds by an average of 2 to 3 mph (3 to 5 km/h). Review of the speed predictions for individual sites suggests that the current method for drawing through-movement paths is somewhat conservative, with drivers not cutting as straight a path as the method suggests. In addition, circulating speeds may be influ-

Table 51. Left-turn-movement circulating speed prediction error.

	Mean Deviation	Root Mean Square Error
Average (Mean) Speed	2.1 mph (3.4 km/h)	2.6 mph (4.2 km/h)
85th-Percentile Speed	0.2 mph (0.3 km/h)	1.7 mph (2.7 km/h)
95th-Percentile Speed	–1.0 mph (–1.6 km/h)	2.4 mph (3.9 km/h)

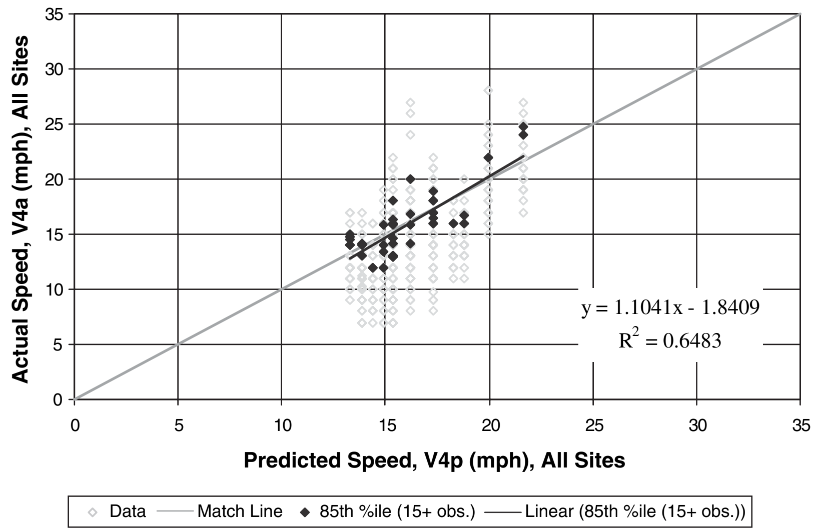

Figure 40. Actual vs. predicted values for left-turn-movement circulating speeds, all sites.

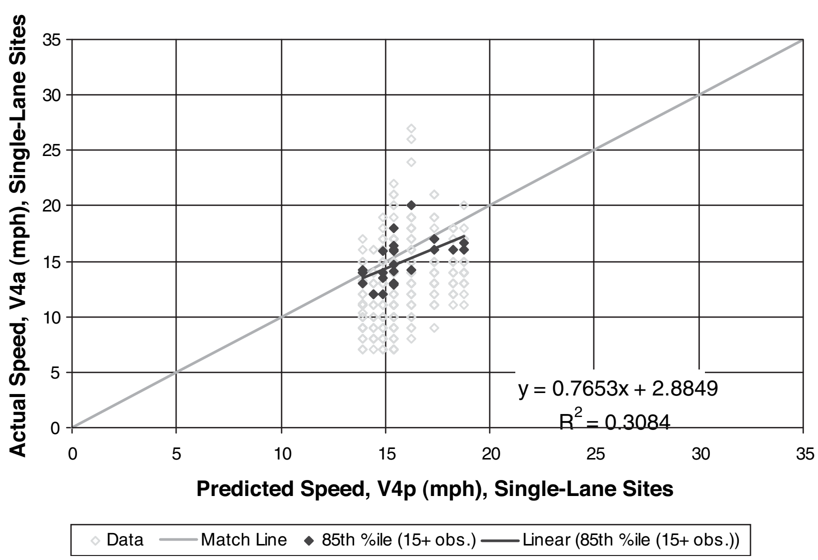

Figure 41. Actual vs. predicted values for left-turn-movement circulating speeds, single-lane sites.

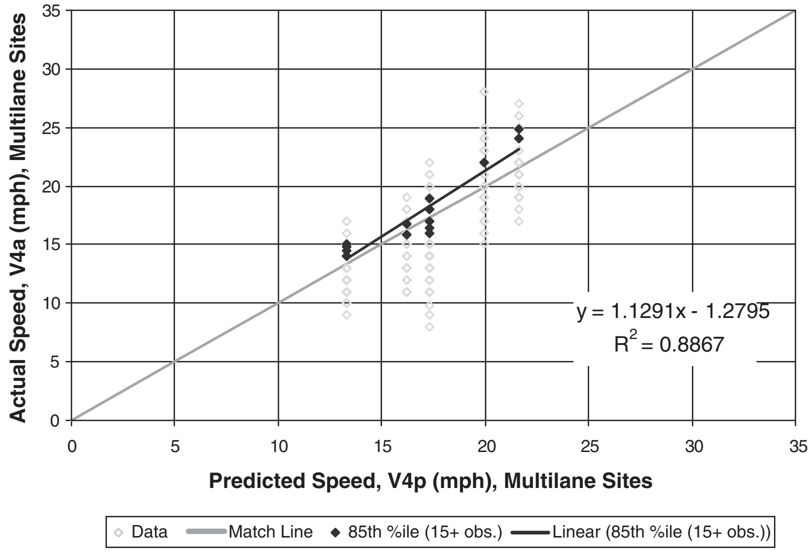

Figure 42. Actual vs. predicted values for left-turn-movement circulating speeds, multilane sites.

Table 52. Summary of through-movement circulating speed (V_2) data.

Characteristic	Total
Range of measured speeds	7 – 31 mph (11 – 50 km/h)
Total number of sites	58
Total number of observations	990
Number of sites with 15+ observations	28
Number of observations at sites with 15+ observations	756

enced by hesitation on entry, which, over time, could be reasonably expected to reduce as drivers become more comfortable. Therefore, the current method for estimating V_2 is generally conservative at the present time but reasonable, and no changes are proposed.

Exit Speed

Table 54 summarizes the characteristics of exit speed data, denoted by V_3 for through movements and V_6 for left-turn movements (these two were combined for analysis). As the exhibit shows, 1,480 of the 1,767 observations, or 84%, occur

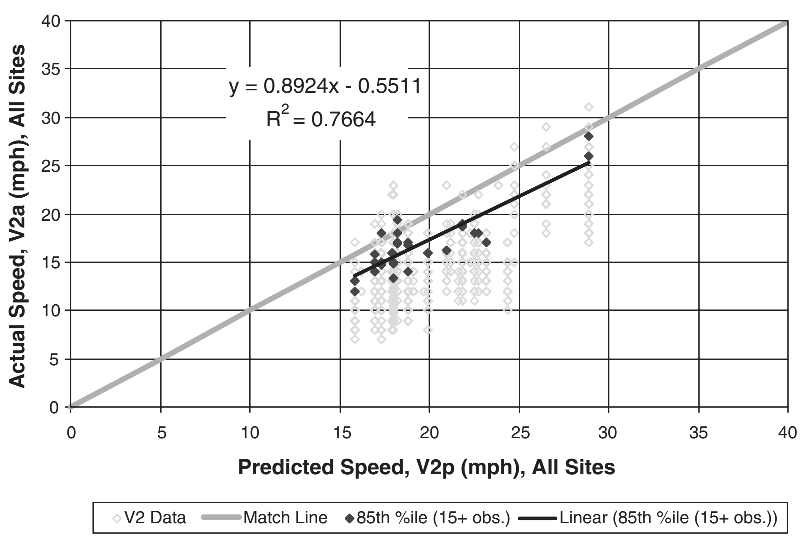

Figure 43. Actual vs. predicted values for through-movement circulating speeds, all sites.

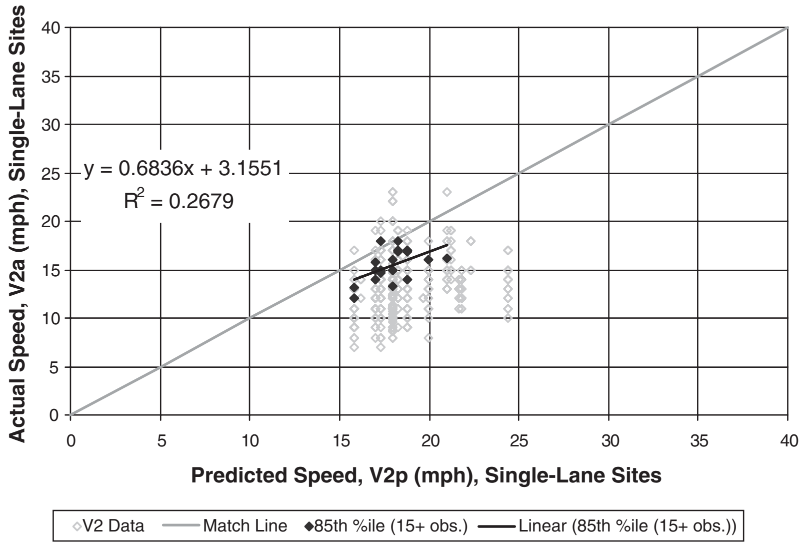

Figure 44. Actual vs. predicted values for through-movement circulating speeds, single-lane sites.

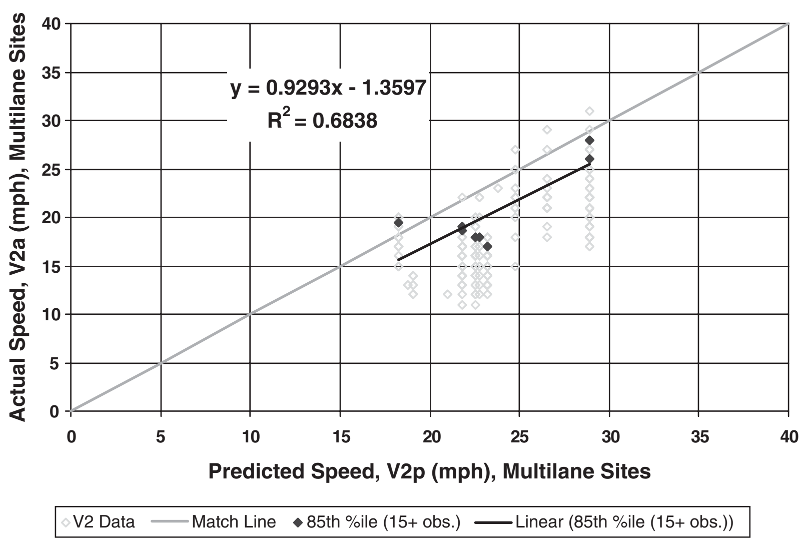

Figure 45. Actual vs. predicted values for through-movement circulating speeds, multilane sites.

Table 53. Through-movement circulating speed prediction error.

	Mean Deviation	Root Mean Square Error
85th Percentile Speed	2.6 mph (4.2 km/h)	3.2 mph (5.1 km/h)

at sites with 15 or more observations. Only 85th-percentile speeds from sites with 15 or more observations were used in the analysis.

Figures 46, 47, and 48 present plots of predicted versus actual exit speeds differentiated by through movements and left-turn movements for all sites, the subset of single-lane sites, and the subset of multilane sites, respectively, along with 85th-percentile values for all sites with 15 or more observations. As can be seen from the figures, the current method for predicting V_3 speeds generally overestimates 85th-percentile speeds, with the error increasing substantially with higher predicted speeds. Note that the cluster of sites with predicted speeds of around 45 mph is arbitrary, as tangential exits with a path radius of infinity were arbitrarily

Table 54. Summary of exit speed data.

Characteristic	Through-Movement Exit Speed, V_3	Left-Turn-Movement Exit Speed, V_6	Total
Range of measured speeds	8 – 37 mph (13 – 60 km/h)	8 – 31 mph (13 – 50 km/h)	8 – 37 mph (13 – 60 km/h)
Total number of sites	56	52	108
Total number of observations	1,084	683	1,767
Number of sites with 15+ observations	38	22	60
Number of observations at sites with 15+ observations	960	520	1,480

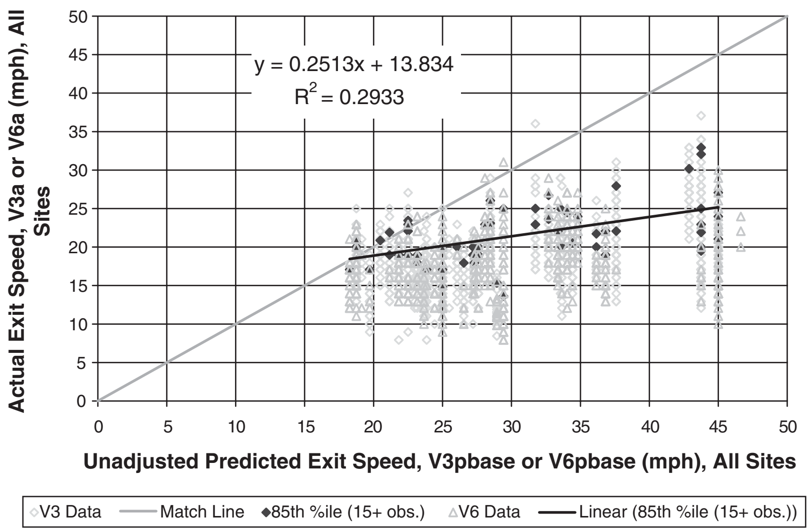

Figure 46. Actual vs. unadjusted predicted values for through-movement and left-turn-movement exit speeds, all sites.

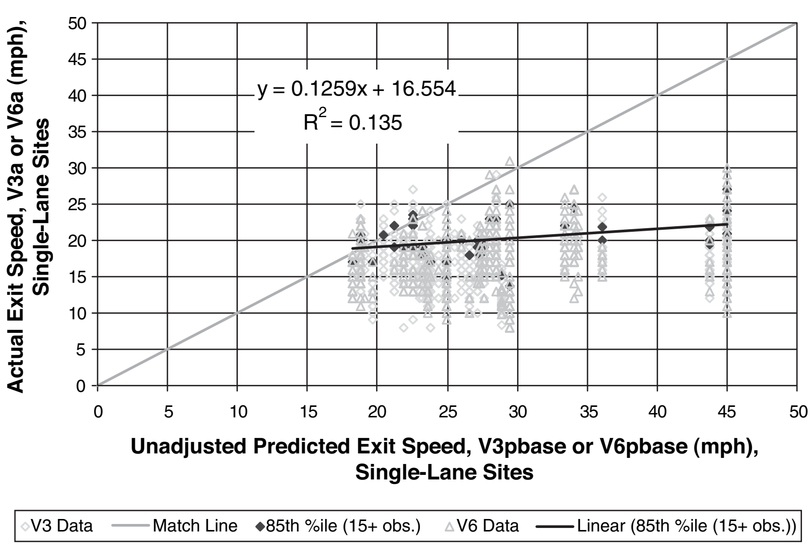

Figure 47. Actual vs. unadjusted predicted values for through-movement and left-turn-movement exit speeds, single-lane sites.

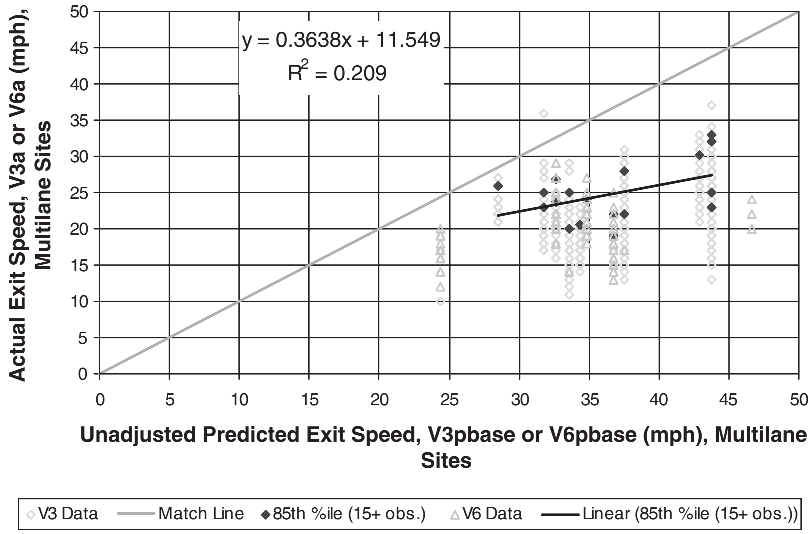

Figure 48. Actual vs. unadjusted predicted values for through-movement and left-turn-movement exit speeds, multilane sites.

assigned a predicted exit speed of 45 mph. However, the cluster of data points with predicted speeds in the 30- to 40-mph range (for which such arbitrary assignments were not made) suggests a significant error in the current method for predicting exit speeds.

To improve the prediction fit for exit speeds, the following formulation is proposed:

$$V_3 = \min\left\{ \begin{array}{c} V_{3pbase} \\ \frac{1}{1.47}\sqrt{(1.47V_2)^2 + 2a_{23}d_{23}} \end{array} \right\} \quad \text{(5-4a, U.S. Customary)}$$

where

V_3 = V_3 speed (mph)
V_{3pbase} = V_3 speed predicted based on path radius (mph)
V_2 = V_2 speed predicted based on path radius (mph)
a_{23} = acceleration along the length between the midpoint of V_2 path and the point of interest along V_3 path = 6.9 ft/s^2 (see text)
d_{23} = distance between midpoint of V_2 path and point of interest along V_3 path (ft)

$$V_3 = \min\left\{ \begin{array}{c} V_{3pbase} \\ 3.6\sqrt{\left(\frac{V_2}{3.6}\right)^2 + 2a_{23}d_{23}} \end{array} \right\} \quad \text{(5-4b, Metric)}$$

where

V_3 = V_3 speed (km/h)
V_{3pbase} = V_3 speed predicted based on path radius (km/h)
V_2 = V_2 speed predicted based on path radius (km/h)
a_{23} = acceleration along the length between the midpoint of V_2 path and the point of interest along V_3 path = 2.1 m/s^2 (see text)
d_{23} = distance between midpoint of V_2 path and point of interest along V_3 path (m)

Vehicle acceleration rates are documented in Exhibit 2-24 in the 2001 AASHTO Policy (*34*), which is based on the findings in *NCHRP Report 270* (*35*). For vehicles in the speed range of 20 to 30 mph (32 to 48 km/h), the latter reference suggests a "design" car acceleration of 0.137 *g*, or 4.4 ft/s^2 (1.3 m/s^2), which is the equivalent of 0 to 60 mph in 20.0 s or 0 to 100 km/h in 20.7 s. Most cars today are capable of straight-line accelerations of at least twice that value, however. Therefore, the data from this study were reviewed to estimate a reasonable acceleration rate exhibited in the field by drivers at roundabout exits.

The average acceleration exhibited by exiting vehicles was estimated by determining the average acceleration between the 85th-percentile field-measured speed for either V_2 and V_3 for through vehicles or V_4 and V_6 for left-turning vehicles, using one half of the distance measured on the plans between the locations where the field measurements were taken (midpoint of the splitter island and the boundary between circulatory roadway and exit, respectively). The use of one half of the distance is arbitrary but reasonable, as it approximates the drivers' ability to accelerate along only approximately half of the vehicular path between (1) the point where they pass the last splitter island prior to exiting and (2) the exit point. This procedure estimated an average acceleration of 6.9 ft/s^2 (2.1 m/s^2), which is the equivalent of 0 to 60 mph in 12.7 s or 0 to 100 km/h in 13.2 s. This acceleration rate appears reasonable and conservative for design purposes.

Using this new formulation, 85th-percentile speeds were plotted against adjusted predicted speeds for all sites, the subset of single-lane sites, and the subset of multilane sites, as shown in Figures 49, 50, and 51, respectively. Estimates for mean deviation and RMSE for both the unadjusted and adjusted predictions are shown in Table 55. The CO03 (Golden, Colorado) site was eliminated from this analysis because of the lack of circulating speed field data to calibrate predicted circulating speeds. As can be seen, the revised prediction is substantially better than the unadjusted prediction; it also eliminates the error associated with estimating a speed for a tangential or nearly tangential exit.

The same data can be presented by site, as shown in Table 56. This table shows that, for most sites, exit speeds appear to be governed by circulating speed and acceleration rather than exit path radius. However, a few single-lane sites—MD02 (Leeds, Maryland), MD03 (Jarrettsville, Maryland), ME01 (Gorham, Maine), OR01 (Bend, Oregon), and WA05 (Sammamish, Washington)—have one or more exit movements whose speeds appear to be governed principally by exit path radius.

Based on this analysis, the proposed exit speed prediction method appears to be a substantial improvement on the current method.

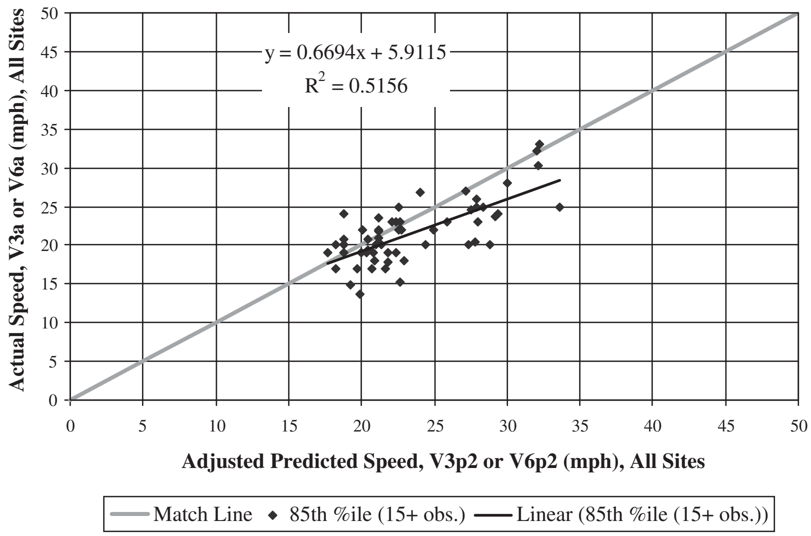

Figure 49. Actual vs. adjusted predicted values for exit speeds, all sites.

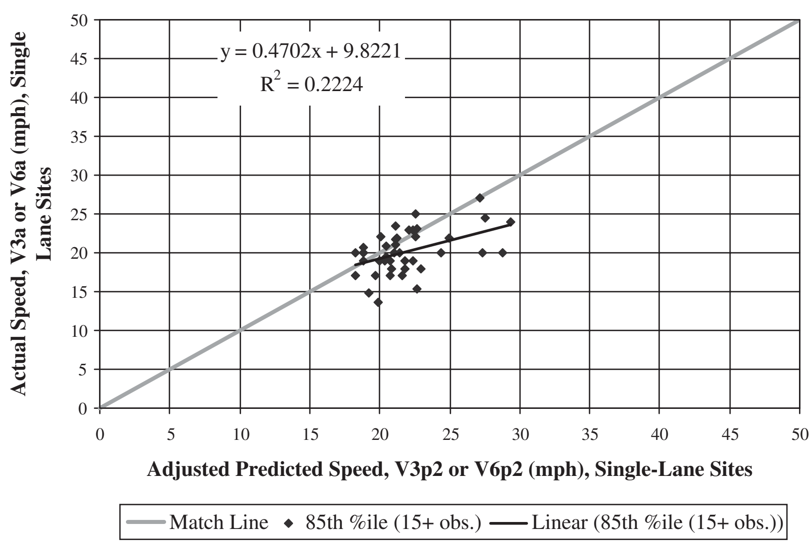

Figure 50. Actual vs. adjusted predicted values for exit speeds, single-lane sites.

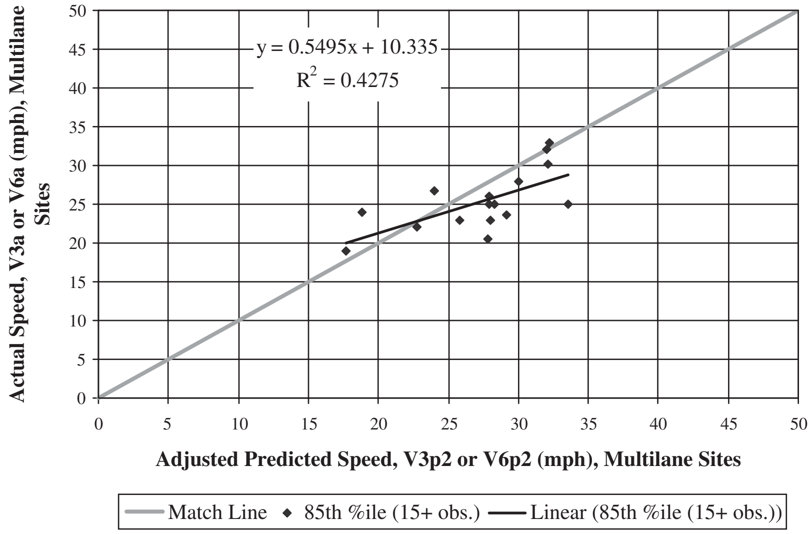

Figure 51. Actual vs. adjusted predicted values for exit speeds, multilane sites.

Table 55. Exit speed prediction error.

V_3 Prediction	Mean Deviation	Root Mean Square Error
Unadjusted		
All Sites	9.1 mph (14.6 km/h)	11.6 mph (18.7 km/h)
Single-Lane Sites	8.3 mph (13.4 km/h)	11.5 mph (18.5 km/h)
Multilane Sites	12.0 mph (19.3 km/h)	13.7 mph (22.0 km/h)
Adjusted		
All Sites	1.8 mph (2.9 km/h)	3.6 mph (5.8 km/h)
Single-Lane Sites	1.8 mph (2.9 km/h)	3.5 mph (5.6 km/h)
Multilane Sites	2.1 mph (3.4 km/h)	4.2 mph (6.8 km/h)

Entry Speed

Table 57 summarizes the characteristics of the entry speed (V_1) data. As the table shows, 1,140 of the 1,503 observations, or 76%, occur at sites with 15 or more observations. These sites with 15 or more observations were used in determining percentiles.

Figures 52, 53, and 54 present plots of predicted versus actual entry speeds differentiated by through movements and left-turn movements for all sites, the subset of single-lane sites, and the subset of multilane sites, respectively, along with 85th-percentile values for all sites with 15 or more observations. As can be seen from the figures, the pattern is similar to that observed for exit speeds, with considerable overestimation of 85th-percentile speeds. As with the exit speeds, the cluster of sites with predicted speeds of around 45 mph is arbitrary, as tangential entries with a path radius of infinity were arbitrarily assigned an entry speed of 45 mph. However, the cluster of data points with predicted speeds in the 30- to 40-mph range (for which such arbitrary assignments were not made) suggests a significant error in the current method for predicting exit speeds.

To improve the prediction fit for entry speeds, the following formulation is proposed:

$$V_1 = \min \begin{Bmatrix} V_{1pbase} \\ \dfrac{1}{1.47}\sqrt{(1.47V_2)^2 - 2a_{12}d_{12}} \end{Bmatrix} \quad \text{(5-5a, U.S. Customary)}$$

where

V_1 = V_1 speed (mph)

V_{1pbase} = V_1 speed predicted based on path radius (mph)

V_2 = V_2 speed predicted based on path radius (mph)

a_{12} = deceleration between the point of interest along V_1 path and the midpoint of V_2 path = -4.2 ft/s^2 (see text)

d_{12} = distance along the vehicle path between the point of interest along V_1 path and the midpoint of V_2 path (ft)

Table 56. Exit speed prediction by site.

Site	Movement Type	Number of Observations	85th-Percentile Field Speed (mph)	Raw Predicted Speed (mph)	Revised Predicted Speed (mph)	Controlling Factor
MD01-N	Through	75	24.0	45.0	29.4	Circ and accel
WA01-E	Through	41	20.0	27.2	18.3	Circ and accel
VT01-S	Through	37	17.0	23.8	20.7	Circ and accel
WA07-N	Left turn	35	18.0	27.3	20.9	Circ and accel
WA01-N	Left turn	34	23.1	28.5	22.7	Circ and accel
NV02-N	Through	33	30.2	42.9	32.2	Circ and accel
CO01-W	Left turn	32	23.0	28.1	22.4	Circ and accel
MI01-W	Left turn	32	24.0	34.8	18.8	Circ and accel
CO02-W	Through	31	20.5	34.4	27.8	Circ and accel
WA07-S	Through	31	23.0	22.5	22.1	Circ and accel
MD06-N	Left turn	31	20.0	25.0	21.0	Circ and accel
WA04-N	Left turn	31	24.5	34.1	27.5	Circ and accel
CO02-E	Through	30	23.7	32.7	29.2	Circ and accel
NV02-E	Through	30	28.0	37.6	30.1	Circ and accel
WA03-E	Through	30	18.0	26.5	22.9	Circ and accel
VT01-N	Left turn	30	13.7	29.5	19.9	Circ and accel
MD01-S	Through	29	20.0	26.0	21.4	Circ and accel
OR01-N	**Through**	**29**	**20.8**	**20.5**	**20.5**	**Exit path radius**
MD07-N	Left turn	29	22.0	33.4	20.1	Circ and accel
OR01-W	**Left turn**	**29**	**20.0**	**18.8**	**18.8**	**Exit path radius**
WA03-E	Left turn	29	19.0	23.2	20.0	Circ and accel
OR01-W	**Through**	**28**	**22.0**	**21.2**	**21.2**	**Exit path radius**
VT03-N	Through	28	23.0	31.8	28.0	Circ and accel
NV02-W	Through	27	32.1	43.8	32.1	Circ and accel
VT03-S	Through	25	25.0	31.8	27.9	Circ and accel
MD03-W	**Through**	**24**	**22.0**	**22.5**	**22.5**	**Exit path radius**
VT03-W	Through	24	26.0	28.5	27.9	Circ and accel
MI01-E	Through	23	23.0	43.8	25.9	Circ and accel
VT01-N	Through	23	15.3	28.9	22.6	Circ and accel
WA04-E	Left turn	23	27.0	45.0	27.1	Circ and accel
VT03-E	Through	22	25.0	33.6	28.3	Circ and accel
WA05-W	**Left turn**	**21**	**17.0**	**18.3**	**18.3**	**Exit path radius**
WA05-W	Through	20	19.0	21.2	20.0	Circ and accel
MD01-E	Left turn	20	21.0	45.0	21.2	Circ and accel
MD04-E	Left turn	19	19.0	36.8	17.7	Circ and accel
ME01-E	Through	18	19.5	43.8	20.4	Circ and accel
WA02-E	Left turn	18	14.9	25.0	19.3	Circ and accel
MD02-E	**Through**	**17**	**19.0**	**21.8**	**21.8**	**Exit path radius**
WA02-W	Through	17	20.0	27.6	27.4	Circ and accel
MD03-N	Through	16	23.5	22.5	21.2	Circ and accel
OR01-S	**Through**	**16**	**20.0**	**18.8**	**18.8**	**Exit path radius**
MD01-N	Left turn	16	19.0	21.8	20.8	Circ and accel
WA01-E	Left turn	16	25.0	29.5	22.5	Circ and accel
VT02-N	Left turn	15	19.0	22.5	18.8	Circ and accel
MD02-N	Through	15	21.8	36.1	21.2	Circ and accel
MD02-S	Through	15	20.0	36.1	24.4	Circ and accel
MD02-W	Through	15	17.9	23.2	21.8	Circ and accel
MD07-N	Through	15	20.0	43.8	28.8	Circ and accel
MI01-W	Through	15	25.0	43.8	33.6	Circ and accel
NV02-S	Through	15	33.0	43.8	32.2	Circ and accel
WA02-E	Through	15	21.9	43.8	25.0	Circ and accel
WA03-S	Through	15	19.0	27.2	22.4	Circ and accel
WA03-W	Through	15	17.0	25.0	21.6	Circ and accel
WA05-E	**Through**	**15**	**17.0**	**19.7**	**19.7**	**Exit path radius**
CO02-E	Left turn	15	22.0	36.8	22.7	Circ and accel
CO02-S	Left turn	15	26.8	32.7	24.0	Circ and accel
ME01-N	**Left turn**	**15**	**20.7**	**18.8**	**18.8**	**Exit path radius**
VT02-E	Left turn	15	19.0	27.6	20.3	Circ and accel

Legend: Circ = circulating speed; accel = acceleration

Table 57. Summary of entry speed (V_1) data.

Characteristic	Through-Movement Entry Speed, V_1	Left-Turn-Movement Entry Speed, V_{1L}	Total
Range of measured speeds	8 – 35 mph (13 – 56 km/h)	8 – 30 mph (13 – 48 km/h)	8 – 35 mph (13 – 56 km/h)
Total number of sites	61	63	124
Total number of observations	927	576	1,503
Number of sites with 15+ observations	34	21	55
Number of observations at sites with 15+ observations	738	402	1,140

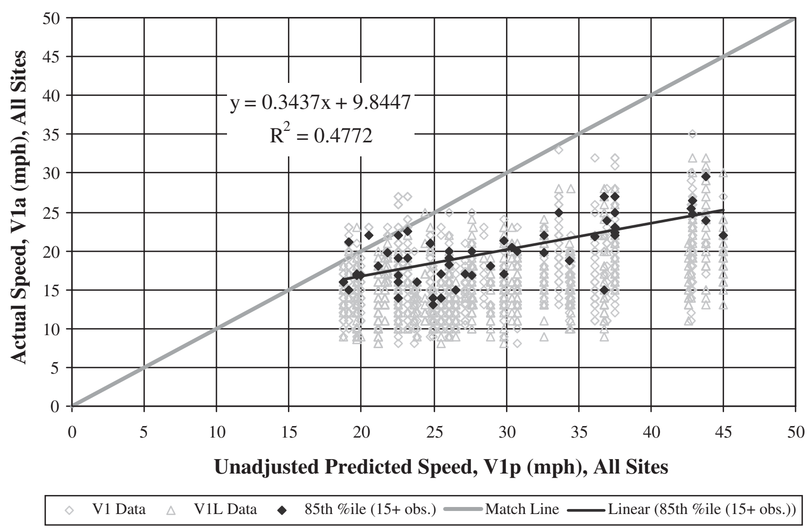

Figure 52. Actual vs. unadjusted predicted values for entry speeds, all sites.

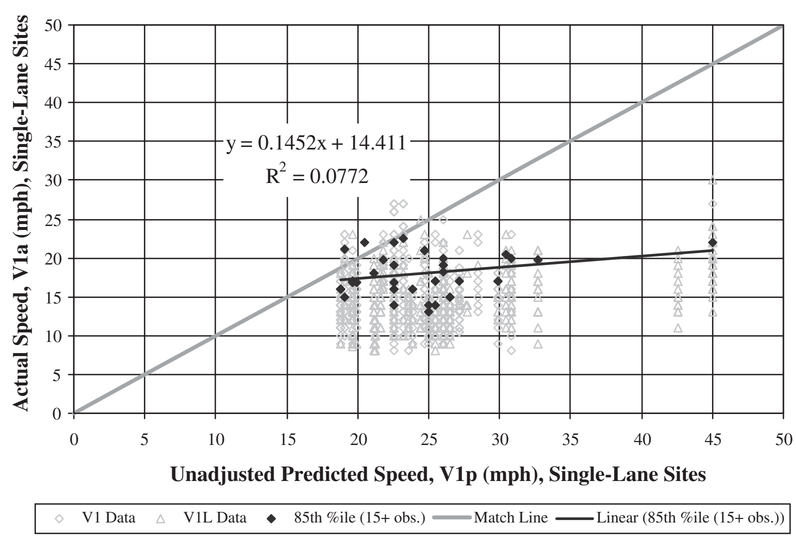

Figure 53. Actual vs. unadjusted predicted values for entry speeds, single-lane sites.

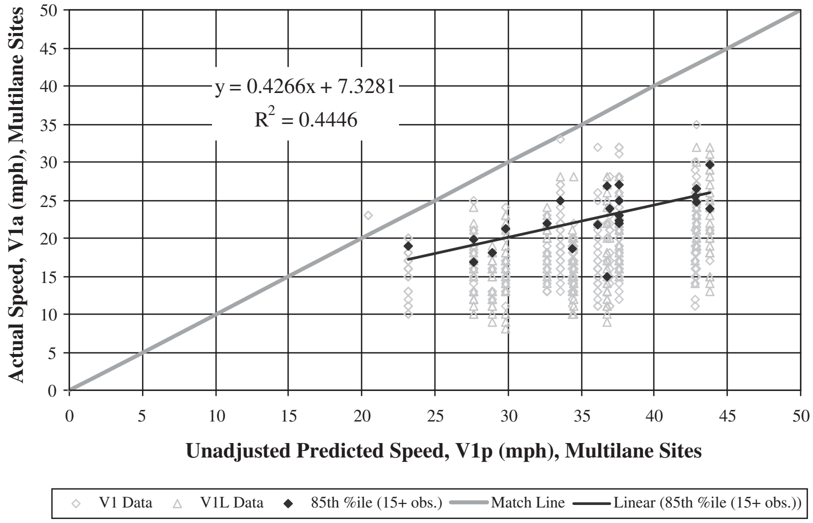

Figure 54. Actual vs. unadjusted predicted values for entry speeds, multilane sites.

$$V_1 = \min\left\{\begin{array}{c} V_{1pbase} \\ 3.6\sqrt{\left(\dfrac{V_2}{3.6}\right)^2 - 2a_{12}d_{12}} \end{array}\right\} \quad \text{(5-5b, Metric)}$$

where

V_1 = V_1 speed (km/h)

V_{1pbase} = V_1 speed predicted based on path radius (km/h)

V_2 = V_2 speed predicted based on path radius (km/h)

a_{12} = deceleration between the point of interest along V_1 path and the midpoint of V_2 path = -1.3 m/s^2 (see text)

d_{12} = distance along the vehicle path between the point of interest along V_1 path and the midpoint of V_2 path (m)

The 2001 AASHTO Policy recommends a deceleration rate of -11.2 ft/s^2 (-3.4 m/s^2) when calculating stopping sight distance (*34*). In addition, a deceleration rate of -10 ft/s^2 (-3.0 m/s^2) is commonly assumed when calculating clearance intervals for signalized intersections (*36*). Deceleration under either of those conditions, however, is likely to be higher than at the approach to a roundabout, because the need to decelerate for an object in the roadway or for a change in signal indication is less predictable than the need to decelerate upon entry into a roundabout. Therefore, it seems reasonable that the deceleration for drivers anticipating a slower circulating speed will be more gradual than that used for signalized intersections.

The average deceleration exhibited by entering vehicles was estimated by determining the average deceleration between the 85th-percentile field-measured speed for either V_1 and V_2 for through vehicles or V_{1L} and V_4 for left-turning vehicles, using one half of the distance measured on the plans between the locations where the field measurements were taken (entry-circulatory roadway boundary and splitter island, respectively). As discussed previously for exit speeds, the use of one half of the distance is arbitrary but reasonable. This procedure estimated an average deceleration of -4.2 ft/s^2 (-1.3 m/s^2). This deceleration rate appears reasonable for design purposes.

Using this new formulation, 85th-percentile speeds were plotted against adjusted predicted speeds for all sites, the subset of single-lane sites, and the subset of multilane sites, as shown in Figures 55, 56, and 57, respectively. Estimates for RMSE for both the unadjusted and adjusted predictions are shown in Table 58. The CO03 (Golden, Colorado) and WA02 (Gig Harbor, Washington) sites were eliminated from this analysis because of the lack of circulating speed field data to calibrate predicted circulating speeds. As can be seen, the revised prediction is substantially better than the unadjusted prediction.

The same data can be presented by site, as shown in Table 59. This table shows that, for most sites, entry speeds appear to be governed by deceleration into an anticipated circulating speed rather than an entry speed governed by entry path radius alone. However, a few sites—MD01 (Bel Air, Maryland), MD03 (Jarrettsville, Maryland), ME01 (Gorham, Maine), and OR01 (Bend, Oregon) among the single-lane sites and MI01 (Okemos, Michigan) among the multilane sites—have one or more entry movements whose speeds appear to be governed principally by entry path radius.

Based on this analysis, the proposed entry-speed prediction method appears to be a substantial improvement on the

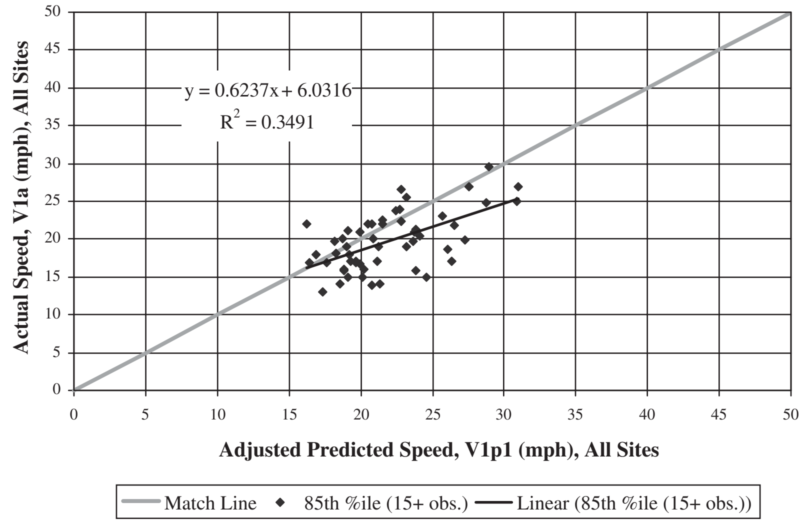

Figure 55. Actual vs. adjusted predicted values for entry speeds, all sites.

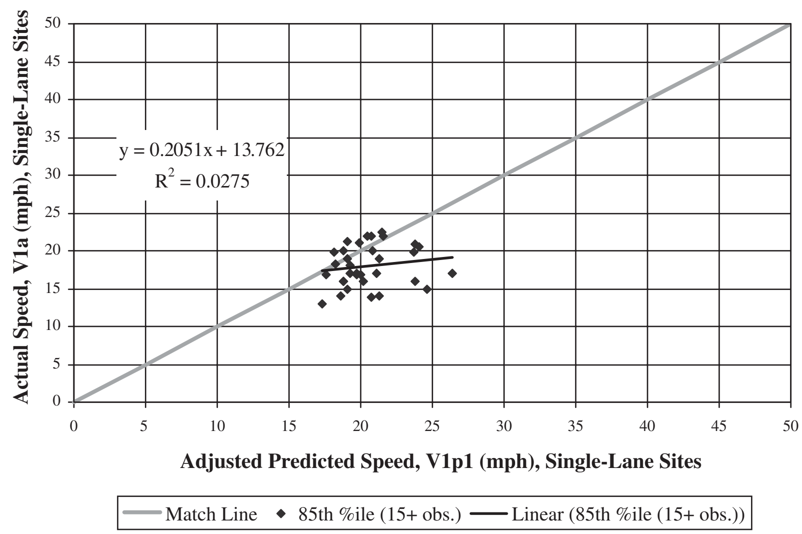

Figure 56. Actual vs. adjusted predicted values for entry speeds, single-lane sites.

current method. However, given the hesitancy currently exhibited by drivers under capacity conditions, the observed entry speeds may increase over time after drivers acclimate further. Therefore, the research team believes that an analyst should be cautious when using deceleration as a limiting factor when establishing entry speeds for design. Furthermore, the research team believes that a good design should rely more heavily on controlling the entry path radius as the primary method for controlling entry speed, particularly for the fastest combination of entry and circulating path (typically the through movement).

Another factor that may influence entry speeds is the amount of available intersection sight distance. The research team gave this hypothesis a cursory evaluation but was unable to pursue it in detail. Additional research on the effect of sight distance on entry speed is recommended.

Conclusions and Recommendations

Based on the analysis presented herein, the following conclusions can be made.

- Current speed prediction methods for predicting 85th-percentile circulating speeds appear to be reliable. Speed prediction is better for movements that track closely around the central island, such as left-turn paths, than for those that are influenced by the central island but do not precisely track around it, such as through-movement paths.

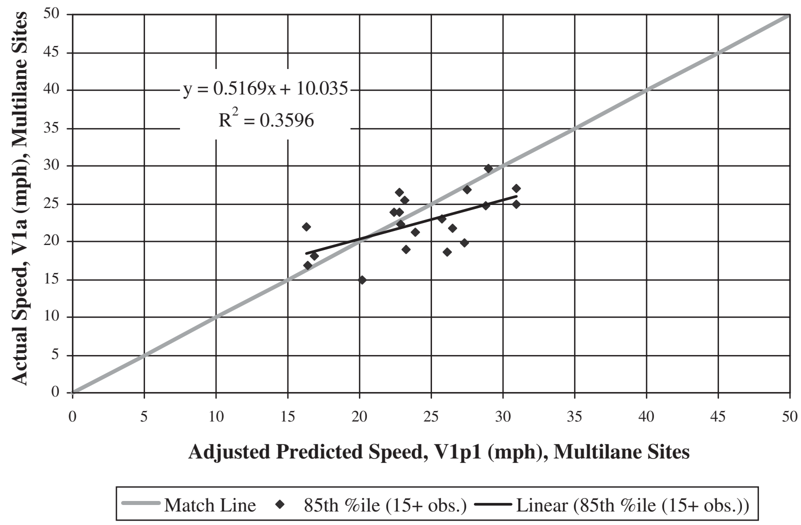

Figure 57. Actual vs. adjusted predicted values for entry speeds, multilane sites.

Table 58. Entry speed prediction error.

Prediction	Mean Deviation	Root Mean Square Error
Unadjusted prediction		
All Sites	9.1 mph (14.6 km/h)	11.0 mph (17.7 km/h)
Single-Lane Sites	6.6 mph (10.6 km/h)	8.6 mph (13.8 km/h)
Multilane Sites	13.2 mph (21.2 km/h)	14.6 mph (23.5 km/h)
Adjusted prediction		
All Sites	2.0 mph (3.2 km/h)	3.8 mph (6.1 km/h)
Single-Lane Sites	2.6 mph (4.2 km/h)	4.2 mph (6.8 km/h)
Multilane Sites	1.5 mph (2.4 km/h)	4.0 mph (6.4 km/h)

- Current speed prediction methods significantly overestimate entry and exit speeds, particularly for entry paths and exit paths that are tangential or nearly tangential. These prediction methods are significantly improved by incorporating acceleration and deceleration effects as they relate to predicted circulating speeds.

Pedestrian Analysis

Data were collected for 769 pedestrian crossing events that occurred at 10 legs, which were distributed among seven roundabouts. The overwhelming majority of these crossings involved adults traversing the crosswalk at a normal pace. There were 6 youth-only crossings and 19 youth-with-adult crossings. There were five crossings on skates or skateboards and five that involved pedestrians walking bicycles. Finally, there were 13 crossings with strollers and 8 crossings in motorized wheelchairs. There were no crossings observed by pedestrians with other diminished capabilities, such as visual impairments.

The analysis of these data was site-based, with a site defined as a leg at a roundabout. Thus, it was possible to have more than one site (leg) at a selected roundabout. Average values for the behaviors at each site were produced and then aggregated to produce overall means. The alternative analysis would have been a pedestrian-based approach, in which all pedestrian events for a given variable were first aggregated before developing means. As previously shown in Table 11, there was also a large range in the number of pedestrian events observed at the various legs. Because the goal of this analysis was to specifically look for geometric or operational features that may contribute to the observed behaviors, the site-based approach was the best choice. By developing site means, any weighting bias due to large sample sizes at one or more sites is removed.

The analysis results are presented in terms of number of lanes on either the entry side or exit side of the leg at the location of the crosswalk. For the 10 legs in the analysis, 5 had one lane in each direction and 5 had two lanes in each direction. In some cases, the number of lanes at the yield line of the

Table 59. Entry speed prediction by site.

Site	Movement Type	Number of Observations	85th-Percentile Field Speed (mph)	Raw Predicted Speed (mph)	Revised Predicted Speed (mph)	Controlling Factor
VT01-N	Through	40	16.0	22.5	20.2	Circ and decel
OR01-S	**Through**	**35**	**15.0**	**19.1**	**19.1**	**Entry path radius**
VT01-S	Through	35	17.1	27.2	19.3	Circ and decel
CO03-E	Through	32	22.0	37.6	—	—
CO03-W	Through	31	25.0	33.6	—	—
WA05-W	Through	31	19.0	22.5	21.3	Circ and decel
CO02-E	Through	30	18.7	34.4	26.1	Circ and decel
MI01-E	**Through**	**30**	**19.0**	**23.2**	**23.2**	**Entry path radius**
CO01-W	Left turn	30	22.0	45.0	21.6	Circ and decel
CO02-W	Through	29	21.8	36.1	26.5	Circ and decel
MD01-S	**Through**	**29**	**16.8**	**20.0**	**20.0**	**Entry path radius**
MD03-E	Through	29	22.0	22.5	20.8	Circ and decel
WA01-E	Left turn	29	19.0	26.0	19.0	Circ and decel
WA07-N	Left turn	29	18.0	21.2	19.2	Circ and decel
WA07-S	Through	28	21.0	24.8	19.9	Circ and decel
MD04-E	Left turn	25	22.0	32.7	16.3	Circ and decel
MD07-N	Left turn	25	20.0	30.8	18.8	Circ and decel
MI01-W	Left turn	23	18.0	28.9	16.8	Circ and decel
VT03-N	Through	22	19.9	27.6	27.3	Circ and decel
WA03-E	Through	21	14.0	25.0	21.3	Circ and decel
MD01-N	**Through**	**20**	**21.2**	**19.1**	**19.1**	**Entry path radius**
MD02-N	Through	20	20.0	26.0	20.8	Circ and decel
VT03-E	Left turn	19	21.3	29.8	23.9	Circ and decel
VT03-S	Left turn	19	22.3	37.6	22.9	Circ and decel
WA04-S	Left turn	18	20.5	30.5	24.1	Circ and decel
WA01-E	Through	17	18.2	26.0	18.3	Circ and decel
NV03-W	Left turn	17	29.6	43.8	29.0	Circ and decel
MD03-S	Through	16	22.5	23.2	21.5	Circ and decel
NV02-E	Through	16	24.8	42.9	28.8	Circ and decel
NV02-S	Through	16	25.0	37.6	30.9	Circ and decel
OR01-N	**Through**	**16**	**17.0**	**19.7**	**19.7**	**Entry path radius**
MD07-W	Left turn	16	14.0	25.5	18.6	Circ and decel
NV02-N	Left turn	16	26.5	42.9	22.8	Circ and decel
WA05-E	Left turn	16	19.8	21.8	18.1	Circ and decel
KS01-E	Through	15	25.5	42.8	23.2	Circ and decel
KS01-W	Through	15	23.9	37.0	22.8	Circ and decel
MD02-S	Through	15	20.9	24.8	23.8	Circ and decel
MD03-N	**Through**	**15**	**22.0**	**20.5**	**20.5**	**Entry path radius**
MD07-S	Through	15	15.0	26.5	24.6	Circ and decel
ME01-W	**Through**	**15**	**15.9**	**18.8**	**18.8**	**Entry path radius**
NV02-W	Through	15	27.0	37.6	31.0	Circ and decel
OR01-E	**Through**	**15**	**16.0**	**18.8**	**18.8**	**Entry path radius**
VT03-S	Through	15	23.0	37.6	25.7	Circ and decel
WA01-W	Through	15	17.0	25.5	21.1	Circ and decel
WA02-E	Through	15	15.9	23.8	23.8	Circ and decel
WA02-W	Through	15	17.0	29.8	26.4	Circ and decel
WA03-N	Through	15	13.9	22.5	20.8	Circ and decel
CO02-S	Left turn	15	14.9	36.8	20.2	Circ and decel
KS01-S	Left turn	15	23.8	43.8	22.4	Circ and decel
MD04-S	Left turn	15	16.9	27.6	16.4	Circ and decel
NV03-S	Left turn	15	26.9	36.8	27.5	Circ and decel
OR01-W	**Left turn**	**15**	**16.9**	**19.7**	**19.7**	**Entry path radius**
VT01-N	Left turn	15	16.9	22.5	17.6	Circ and decel
WA03-E	Left turn	15	13.0	25.0	17.3	Circ and decel
WA04-E	Left turn	15	19.8	32.7	23.7	Circ and decel

Legend: Circ = circulating speed; decel = deceleration roundabout did not match the number of lanes at the location of the crosswalk. For example, one leg had two lanes at the yield line for motor vehicles but only a single lane at the point where the crosswalk was located. This leg was classified as a one-lane site in the analysis. The results are also stratified by entry side and exit side, both in terms of where the pedestrian initiated the crossing and in terms of the behaviors on each side. There have been a number of concerns raised over the safety of pedestrians within the exit lanes specifically. The results were stratified to determine if there are differences in the behaviors on each side of the crossing.

The analysis results are presented in the following sections:

- Pedestrian Crossing Behaviors
- Motorist Behaviors
- Pedestrian-Motor Vehicle Conflicts
- Pedestrian Crossing/Wait Times
- Comparison to Other Intersection Types

For further breakdowns of the figures provided, Appendix K includes several tables with results on a site-by-site basis. Appendix L includes images to help describe some of the maneuvers observed.

Pedestrian Crossing Behaviors

The examination of pedestrian crossing behaviors revealed that the majority of crossings involved no interaction with a motor vehicle, where interaction is defined as the pedestrian either accepting or rejecting a gap when a vehicle was present. Figure 58 shows the percentages of crossings requiring interaction with a vehicle on either the entry side or exit side as a function of where the crossing was initiated. For one-lane sites, there was virtually no difference in the level of interaction between the entry and exit sides if starting position is not considered: 27% on the entry side and 26% on the exit side. For crossings that started on the entry side, 32% of the crossings required interaction on the entry side (the first stage of a two-stage crossing), while only 27% of such crossings required interaction on the exit side (the second stage of a two-stage crossing). For crossings that started on the exit side, the numbers are reversed: 26% of the crossings required interaction on the exit side, while 22% required interaction on the entry side.

This same pattern was true for exit starts for two-lane sites. Almost half (45%) of such starts required interaction on the exit side, as opposed to 22% requiring interaction on the entry side. For entry side starts on two-lane sites, the numbers were reversed: 26% of these starts required interaction on the entry side, while 33% required interaction on the exit side. Overall, the level of interaction was greater on the exit side (39%) compared to the entry side (24%) when the starting position is not considered.

For those pedestrians who did interact with vehicles and ultimately crossed the leg, their behaviors were categorized as one of the following:

- **Normal:** Pedestrian crossed the street at a normal pace (walking speed). None of the following behaviors were observed, and the vehicle yielded.
- **Hesitates:** Pedestrian hesitated on the curb or splitter island because of an approaching vehicle. Most often, the hesitation occurred while the pedestrian made visual or other contact with the driver. Once this communication was made and the vehicle began slowing, the pedestrian would then proceed with the crossing.

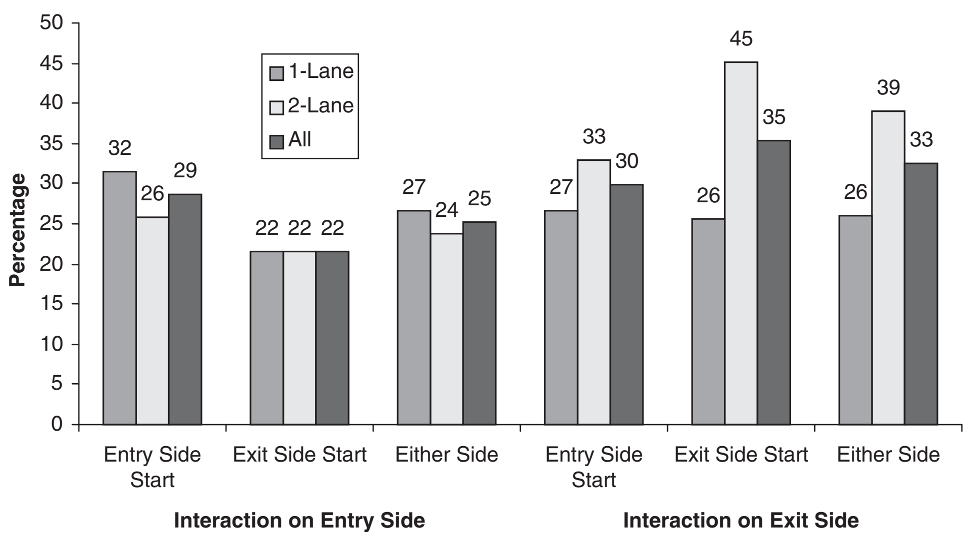

Figure 58. Percentage of crossings in which the pedestrian is required to interact with one or more motor vehicles.

- **Retreats:** Pedestrian began crossing and then retreated to the curb or splitter island because of an approaching vehicle.
- **Runs:** Pedestrian ran across the leg because of an oncoming vehicle. Note that running did not indicate that a conflict was imminent; it simply indicates a choice that was made by the pedestrian. Conflicts are covered in a later section of this report.

Figures 59 and 60 show the distributions of these behaviors based on whether the crossing began on the entry side or exit side, respectively. The one behavior that did not occur for any of the observed crossings was a retreat to the curb or splitter island.

For crossings that began on the entry side (see Figure 59), approximately 60% of the crossings were considered to be normal when considering all sites and either side of the crossing. The most observed non-normal behavior on the entry side was hesitation: 25% of the pedestrians hesitated when crossing one lane, while 40% of the pedestrians hesitated when crossing two lanes. The hesitation on the splitter island (captured under the exit side behavior) was much lower at 9% and 12% for crossing one and two lanes, respectively.

The other behavior that was observed was running. For entry-side starts, the running behavior was much more prevalent on the exit side: 39% of the pedestrians completed their crossings by running across one-lane sites, while only 19% were observed to run across the exit side on two-lane sites. For both site types, the level of running was much lower on the entry side: 12% and 3% for one-lane and two-lane sites, respectively.

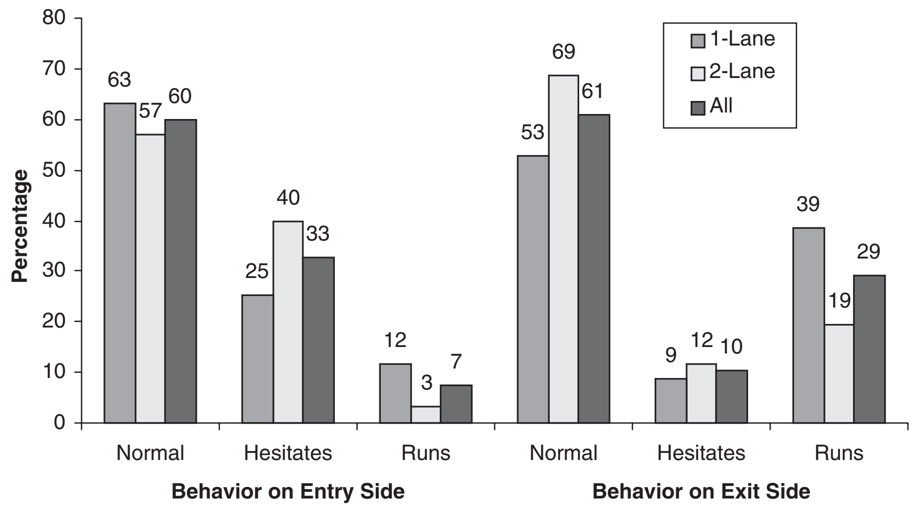

Figure 59. Pedestrian crossing behaviors when a vehicle was present and the crossing began on the entry side.

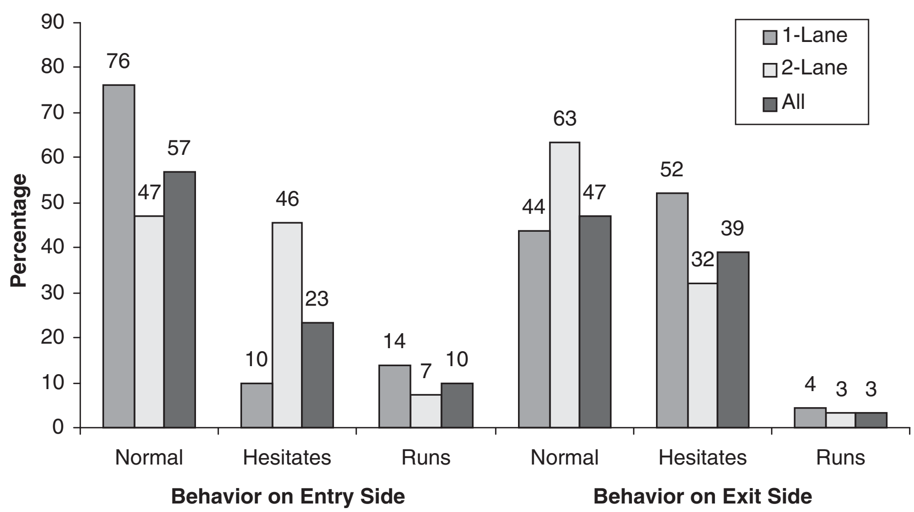

Figure 60. Pedestrian crossing behaviors when a vehicle was present and the crossing began on the exit side.

For crossings that were initiated on the exit side (see Figure 60), the overall percentage of crossings that were coded as normal is lower than what was observed for entry side starts. For one-lane sites, 76% of the crossings were normal on the entry side, while only 44% of such crossings were normal on the exit side. For two-lane sites, 47% of the crossings were normal on the entry side, and 63% were considered normal on the exit side. Similar to entry-side starts, the most-observed non-normal behavior for exit-side starts was hesitation. For one-lane sites, 52% of the pedestrians hesitated on the exit side (which is the starting side in this case), while only 10% hesitated on the splitter island. For two-lane sites, 32% of the pedestrians hesitated on the exit side, and 46% hesitated on the splitter island. Crossings starting on the exit side of two-lane sites are the only instances in which the hesitation on the splitter island exceeded the hesitation on the initial curb.

With respect to the behavior of running, the patterns for exit-side starts were similar to those for entry-side starts, i.e., pedestrians ran more often on the second stage of the crossing. At one-lane sites, 4% of the pedestrians ran across the exit side, while 14% ran across the entry side. At two-lane sites, 3% ran across the exit side, while 7% ran across the entry side.

Another behavior that was observed for pedestrian crossings was whether the crossing was made within or outside the boundaries of the crosswalk. The overwhelming majority of crossings were made within the crosswalk boundaries (see Figure 61). However, 17% of the crossings at one-lane sites and 12% at two-lane sites occurred completely outside the crosswalk lines. For these crossings, 38% involved pedestrians who hesitated when crossing the entry side, while 27% did the same on the exit side. An additional 8% and 9% of these crossings involved pedestrians who ran across the entry and exit sides, respectively. At one of the one-lane sites (MD05, Towson, MD), half of the pedestrians observed to cross out of the crosswalk used a center turn lane that was located upstream of the splitter island as a refuge area.

Motorist Behaviors

For each pedestrian event captured, the behavior of the motorist was also recorded. These behaviors were collapsed into the following three categories for the analysis:

- **Active yield:** The motorist slowed or stopped for a crossing pedestrian or a pedestrian waiting on the curb or splitter island to cross. The pedestrian was the only reason the motorist stopped or slowed.
- **Passive yield:** The motorist yielded to the pedestrian but was already stopped for another reason. This situation occurred most often when there was a queue of vehicles waiting to enter the roundabout or when the vehicle was already stopped for a prior pedestrian crossing event.
- **Did not yield:** The motorist did not yield to a crossing pedestrian or a pedestrian waiting on the curb or splitter island to cross.

Figure 62 shows the yielding behavior results when the pedestrian crossing is initiated on the entry side. For one-lane sites, 15% of the motorists did not yield to the pedestrian on either the entry or exit side. The remainder of the exit-side vehicles actively yielded. The remainder of the entry-side vehicles included 20% that were classified as passively yielding. For two-lane sites, the percentage of non-yielding vehicles increases to 33% on the entry side and 45% on the exit side. For those vehicles that did yield, 9% and 2% were classified as passive yield for the entry and exit sides, respectively.

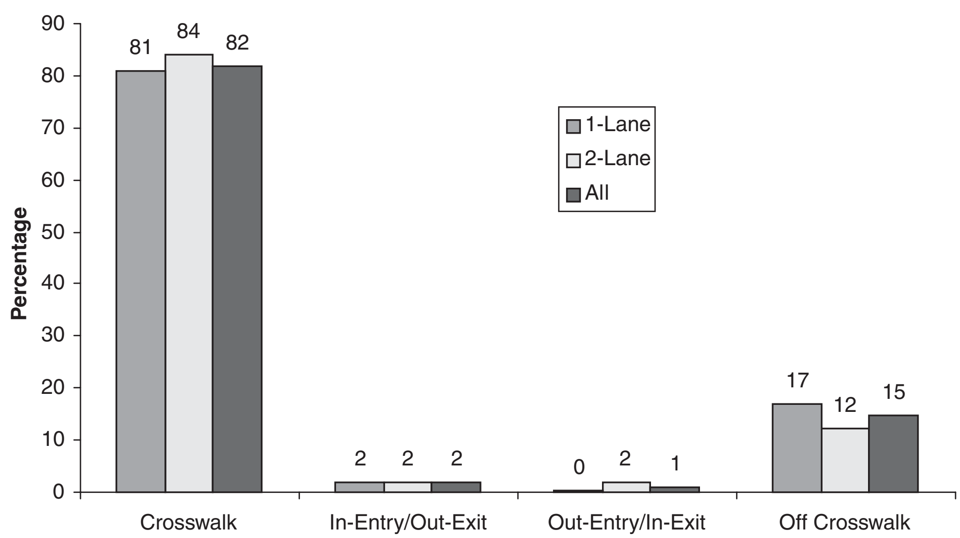

Figure 61. Pedestrian behaviors related to position in the crosswalk when crossing.

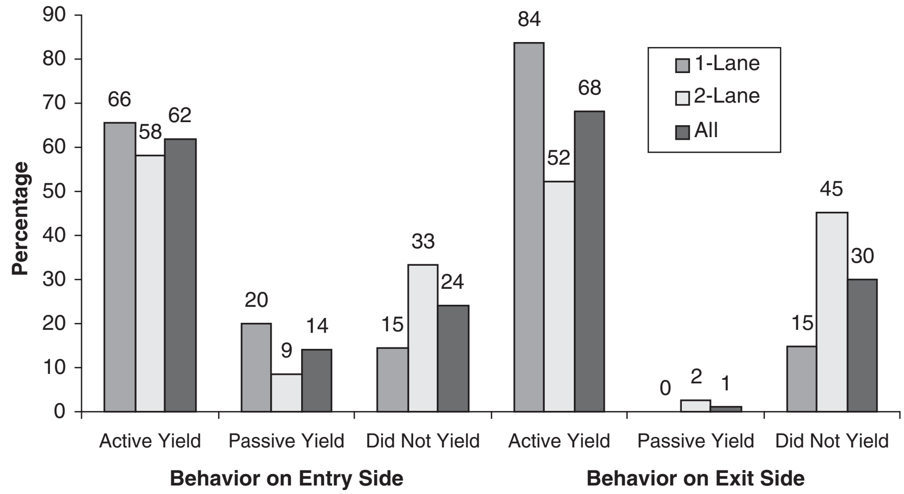

Figure 62. Yielding behavior of motorists when the pedestrian crossing begins on the entry side.

The motorist yielding behavior results for pedestrian crossings that started on the exit side are shown in Figure 63. For one-lane sites, 29% of the motorists did not yield to the pedestrian on the exit side, and 10% did not yield on the entry side. Of the vehicles that did yield, 14% and 18% passively yielded on the exit and entry sides, respectively. For two-lane sites, the percentage of vehicles not yielding increased, just as it did for the entry-start crossings. The percentage of non-yielding vehicles increased to 62% on the exit side and 33% on the entry side. For those vehicles that did yield, 9% and 7% were classified as passive yield for the exit and entry sides, respectively.

Overall, when looking at the entire two-stage crossing, approximately 27% of the motorists did not yield to crossing or waiting pedestrians that started crossing from the entry side. The percentage of non-yielding motorists increases to 34% for crossings initiated on the exit side. In addition, the lack of yielding on two-lane sites (43%) is substantially worse than on one-lane sites (17%).

Yielding behavior was also observed for sites in different regions of the country, specifically east versus west. Sites from Florida, Maryland, and Vermont were included in the east group, while the west group included locations from Washington, Nevada, and Utah. Each region was balanced to include two one-lane sites and two two-lane sites. Motorist non-yielding behavior was observed more often at the eastern sites (35%) compared to the western sites (27%). The difference was most pronounced on the exit side, where the east and west non-yield percentages were 48% and 29%, respectively. These observations suggest that perhaps driver

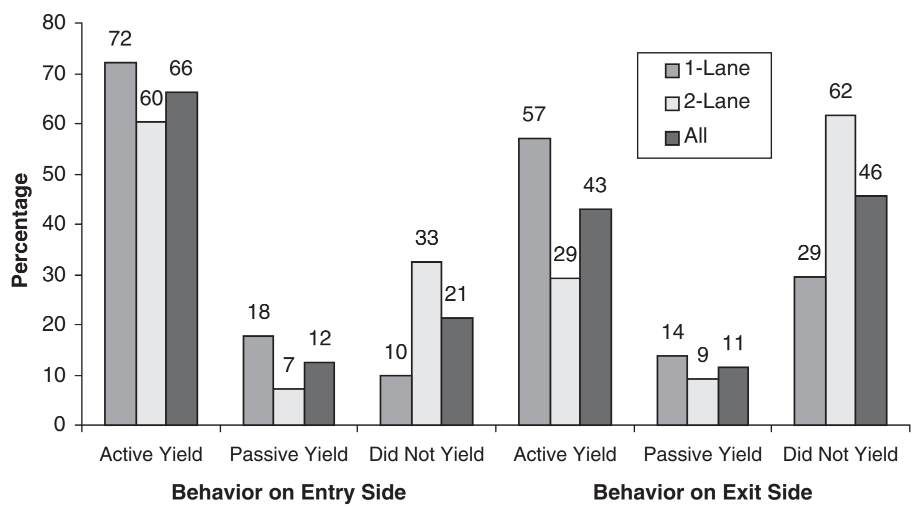

Figure 63. Yielding behavior of motorists when the pedestrian crossing begins on the exit side.

behavior with respect to pedestrians may be influenced by more than just the design.

Pedestrian-Motor Vehicle Conflicts

One of the surrogate measures of safety for pedestrians is a conflict with a motor vehicle. In this study, as well as many others, a conflict was defined as an interaction between a pedestrian and motorist in which one of the parties had to suddenly change course and/or speed to avoid a crash. During the 769 pedestrian crossing events, only four conflicts were observed (0.5%). Two of these conflicts occurred at one one-lane site and two occurred at a different one-lane site. The conflicts were also divided between the entry and exit sides (one each) at each site. Each conflict is further described below:

- **Conflict 1 – WA03-S (Bainbridge Island, Washington) entry side:** Pedestrian emerges from a shadow; approaching vehicle brakes hard (Figure 64).
- **Conflict 2 – WA03-S (Bainbridge Island, Washington) exit side:** Pedestrian crosses exit side; exiting vehicle brakes hard and swerves left (Figure 65).
- **Conflict 3 – MD05SW-S (Towson, Maryland) entry side:** Pedestrians come from behind a stopped bus; approaching vehicle brakes hard (Figure 66).
- **Conflict 4 – MD05SW-S (Towson, Maryland) exit side:** Pedestrian crosses exit side close to circulating lanes; exiting vehicle brakes hard (Figure 67).

Rather than view conflicts in absolute numbers, another approach is to calculate a conflict rate based on opportunities. In this study, an opportunity was defined as any time a pedestrian was either waiting to cross or crossing the leg *and* a motor vehicle was in the vicinity of the pedestrian. To avoid

Figure 64. Pedestrian Conflict 1 at WA03-S (Bainbridge Island, WA).

Figure 65. Pedestrian Conflict 2 at WA03-S (Bainbridge Island, WA).

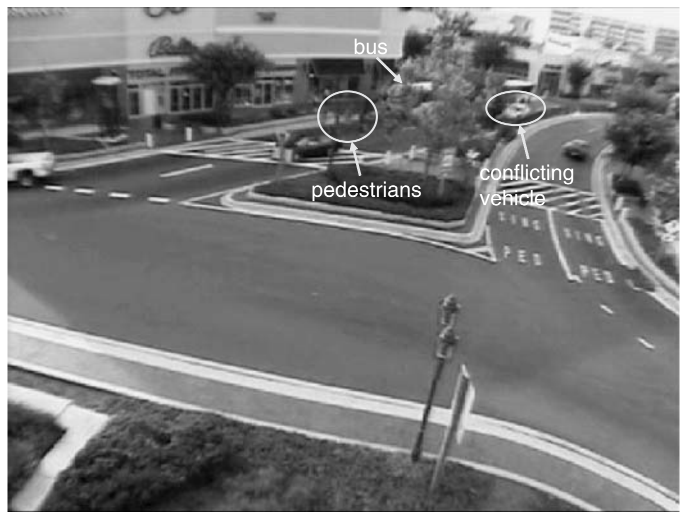

Figure 66. Pedestrian Conflict 3 at MD05SW-S (Towson, MD).

Figure 67. Pedestrian Conflict 4 at MD05SW-S (Towson, MD).

Table 60. Pedestrian-vehicle conflicts per 1000 opportunities.

Number of Lanes	Opportunities	Conflicts	Conflict Rate/ 1000 Opportunities
1	707	2	2.8
2	1,011	2	2.0
All	1,718	4	2.3

a conflict, both parties had to respond correctly. The pedestrian had to reject gaps when the motorist did not yield, and the motorist had to yield when the pedestrian was crossing. Table 60 shows the rates of pedestrian-vehicle conflicts across all study sites. The rate for one-lane sites (2.8 conflicts/1000 opportunities) was slightly greater than the rate for two-lane sites (2.0 conflicts/1000 opportunities).

Similar rates were calculated for the two sites where the conflicts were observed. The one-lane site, WA03-S (Bainbridge Island, Washington), had a rate of 7.1 conflicts/1000 opportunities. The two-lane site, MD05SW-S (Towson, Maryland), had a rate of 15.0 conflicts/1000 opportunities. Both of these values are much greater than the mean rates and may provide an indication of a potential safety concern at these sites.

Pedestrian Crossing/Waiting Times

Times were recorded for each pedestrian event from the point at which the pedestrian arrived until s/he completed the crossing. These data allowed for the derivation of the following time-based measures:

- **Initial waiting time:** The difference in time between the point of arrival and the time at which the pedestrian began crossing the street.
- **Splitter time:** The difference in time between the arrival time and the departure time at the splitter island. This time included both the time to traverse the splitter island and any time spent waiting on the island.
- **Crossing time:** The difference in the time at the end of the crossing and the time at which the pedestrian began crossing the street.

The values for each of these measures are shown in Figure 68 for one-lane, two-lane, and all sites. For one-lane sites, the average waiting time was about 1.3 s, irrespective of starting location for the crossing. For two-lane sites, the same value was derived for crossings starting on the entry side. However, crossings initiated on the exit side on two-lane sites required an average waiting time of 2.9 s. Splitter times and crossing times varied little on the basis of starting position. The longer splitter times on two-lane sites (4.7 s mean) compared to one-lane sites (2.0 s mean) was more likely a function of the size of the islands and the time required to traverse it than a function of actual waiting time on the island. Almost all pedestrians traversed the splitter island without stopping; the average waiting time (across all sites) on the splitter island was 0.4 s. The mean crossing time for a one-lane site was 9.0 s and included time to cross the entry lane, splitter island, and exit lane. For two-lane sites, the mean crossing time was 14.4 s. The crossing times are discussed further in the next section of the report as it relates to the pace of the crossing.

Comparison to Other Intersection Types

The analysis of the pedestrian data collected at roundabouts included an examination of crossing behaviors,

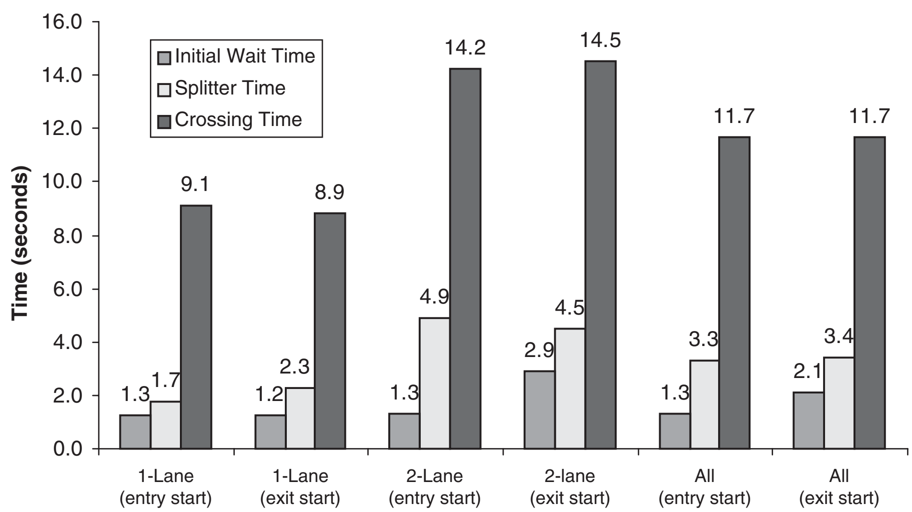

Figure 68. Average pedestrian waiting and crossing times.

motorist yielding behaviors, and pedestrian waiting and crossing times. In an effort to provide more insight into these results, the roundabout findings were compared to results from intersections with other types of traffic control that are more commonly found in the United States, such as signalization and stop signs. In an ongoing FHWA-sponsored project titled "Safety Index for Assessing Pedestrian and Bicyclist Safety at Intersections" (*3*), an effort was undertaken to acquire similar types of pedestrian and motorist behavior data at 68 signalized and stop-controlled intersections. Twelve of those sites were on one-way streets and were not included in this comparative analysis; two other sites were also not included in the analysis. For the 54 sites included, a total of 2,881 pedestrian crossing events were observed. This section of the report compares the results for roundabouts from this study and the results for other intersection types from the FHWA study in the following areas:

- Pedestrian crossing behavior
- Motorist behavior
- Pedestrian waiting time
- Pedestrian crossing pace

Comparison of Pedestrian Crossing Behaviors

The FHWA study observed pedestrian crossings at crosswalks and classified their behavior in one of the following categories:

- Went around a vehicle that was blocking crosswalk
- Ran to avoid approaching vehicle
- Stopped while crossing to let vehicle pass
- Aborted crossing; stepped into roadway and then stepped back onto the curb to let vehicle pass
- Proceeded normally across; did not take one of the above actions

As shown in the above list, the behavior categories are similar to the categories used in this research. However, the FHWA study did not collect data on every rejected gap, only those rejected gaps when the pedestrian did something other than stand and wait (i.e., stopped or aborted crossing). Therefore, the results that follow are based on events where there was a vehicle present, which allows for a more accurate comparison. Similar to the analysis for the roundabout research, the FHWA data were summarized on a per-site basis, where mean performance measures were calculated for each site and then averaged together. As with the roundabout analysis, a site is defined as a crossing location or crosswalk; thus, a four-leg intersection could have four sites included in the analysis. At signalized and all-way-stop–controlled intersections, all sites are subject to the same type of traffic control. However, at two-way-stop–controlled intersections, there are two stop-controlled sites and two sites with no control.

The behavior distributions from each study are shown in Tables 61 and 62. Because these were two independent studies, the behavior categories were slightly different. However, there is enough similarity to draw reasonable conclusions. The dominant behaviors for crossings at a roundabout were "normal" (proceeded without stopping) at 58% and "hesitated on curb" at 27%. The dominant behavior for all other intersections was "proceeded normally" at 88%. However, the FHWA study only recorded pedestrian behavior if the pedestrian had started crossing, which means that any hesitation on the curb at these locations was captured within the normal crossing category. Combining the normal crossings and curb hesitations for the roundabout crossings produces a value of 85%, which is essentially equivalent to 88% for normal crossings at standard intersections. As shown in Table 62, there were differences in the percentage of crossings considered to be normal when considering the type of traffic control: signalized (90%), stop-controlled (100%), and no control (70%). The value of 85% for roundabout crossings falls between the values for no control and signalized control and is expected, given that the yield control present at roundabouts falls between these extremes.

Aborted crossings, which were coded as "retreated" in the roundabout effort, were non-existent across all levels of traffic control. Crossings in which the pedestrian stopped after starting did not occur at roundabouts or at stop-controlled

Table 61. Pedestrian behavior at roundabout crosswalks.

Pedestrian Behavior	Occurrence
Normal	58%
Hesitated on curb	27%
Hesitated after start	1%
Stopped after start	0%
Retreated	0%
Ran	14%
Total	*100%*

Table 62. Pedestrian behavior at common intersection-type crosswalks.

Pedestrian Behavior	*Occurrence by Traffic Control on Leg with Crosswalk*			
	Signal	Stop Sign	None	All Types of Control
Proceeded normally	90%	100%	70%	88%
Aborted crossing	0%	0%	0%	0%
Went around blocking vehicle	5%	0%	1%	3%
Ran to avoid	1%	0%	2%	1%
Stopped to let vehicle pass	4%	0%	27%	8%
Total	*100%*	*100%*	*100%*	*100%*

intersections. It did occur for 4% of the crossings at signalized intersections and for 27% of the crossings at uncontrolled intersections. Running behavior was observed much more often at roundabout crossings than at any other type of crossing. However, running at roundabout crossings was observed to occur mainly during the second half of the crossing and was usually done out of courtesy to waiting motorists, as opposed to a behavior that was required to avoid a conflict.

Comparison of Motorist Behaviors

The FHWA study observed motorist behavior during pedestrian crossings and classified it as yielding or not yielding. Data on the yielding behavior were insufficient to categorize the yields as active or passive, as was done in the analysis for this roundabout research. Table 63 shows how the motorist behavior at roundabouts compares to behavior at the three other types of traffic control. Almost half (48%) of the vehicles on uncontrolled legs did not yield to pedestrians. The crossings subject to yield control (roundabouts) resulted in 32% of the motorists not yielding. Finally, the stop-controlled and signalized intersections produced non-yielding vehicles percentages of 4% and 15%, respectively. These results correlate with the pedestrian behaviors previously discussed and reflect the level of traffic control's influence on the yielding behavior of motorists.

Comparison of Pedestrian Waiting Time

Pedestrian waiting time for roundabout crossings was calculated as the amount of time between the arrival at the curb and start of crossing plus the waiting time on the splitter island. Waiting time at signalized, stop-controlled, and uncontrolled sites was calculated as the amount of time between arrival and start of crossing. The traffic control type that resulted in the longest pedestrian waiting time (10.7 s) was signalization (see Table 64). This result is expected, as most pedestrians complied with the signals as opposed to selecting their own gaps. Crossings at sites with no traffic control produced an average waiting time of 3.0 s, while roundabout crossings caused pedestrians to wait for an average of 2.1 s. Crossings at stop-controlled sites resulted in virtually no waiting time (0.3 s) for pedestrians.

Table 63. Motorist behaviors by traffic control type.

Type of Traffic Control	Yielded	Did Not Yield
Roundabout (yield control)	68%	32%
Signal	85%	15%
Stop Sign	96%	4%
None	52%	48%

Table 64. Waiting time by traffic control type.

Type of Traffic Control	Average Waiting Time (s)
Roundabout (yield control)	2.1
Signal	10.7
Stop Sign	0.3
None	3.0

Comparison of Pedestrian Crossing Pace

Pedestrian crossing pace was calculated as the crossing width divided by the average crossing time for each site. This comparison was based on crossing pace rather than total crossing time so that sites with different lane widths and different splitter island and median configurations could be compared. Figure 69 shows the pace comparisons for one-lane and two-lane sites (in each direction), as well as all sites. Overall, the crossing paces were very similar, ranging only from 4.4 to 5.0 ft/s (1.3 to 1.5 m/s). The type of traffic control that is present at a crossing does not appear to produce any practical differences in the walking pace of crossing pedestrians.

Analysis of Findings

This study was undertaken to develop a better picture of pedestrian operations at roundabouts and to gain insight on the interactions between this mode and motor vehicles. Data were collected from 10 sites located at seven roundabouts to answer the questions that were posed in the introduction section of this report. Provided below are answers to these questions on the basis of the analysis conducted in this effort.

What is the yielding behavior of motorists when they encounter a pedestrian who is crossing or waiting to cross? On average across all sites, approximately 30% of the motorists did not yield to pedestrians who were crossing or waiting to cross. In all but one case, the pedestrians were waiting to cross, so there was no imminent risk. There was a difference in this behavior with respect to the entry side versus exit side of the leg being crossed. Motorists did not yield to pedestrians on the entry side 23% of the time, compared to 38% of the time on the exit side. There was also a difference in the yielding behavior depending on where the crossing was initiated. If the pedestrian started crossing from the entry side of the leg, 27% of the motorists did not yield. However, if the crossing began on the exit side, the percentage of motorists not yielding increased to 34%.

Yield behavior also varied with the number of lanes at the crosswalk. The lack of yielding on two-lane sites (43%) was substantially worse than on one-lane sites (17%). The results for two-lane sites showed that 54% of the motorists did not

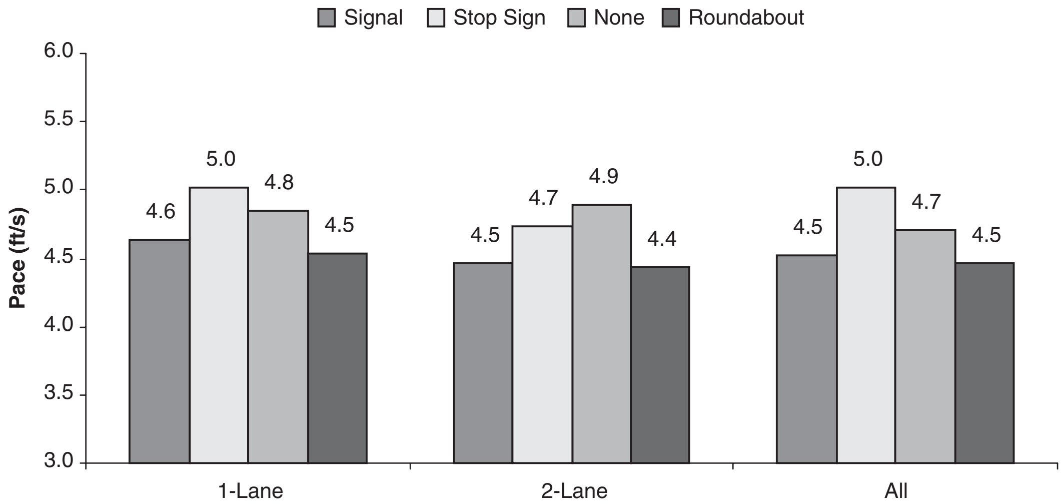

Figure 69. Pedestrian crossing pace by traffic control and number of lanes.

yield on the exit side compared to 33% not yielding on the entry side. For one-lane sites, the exit and entry non-yield percentages were 22% and 13%, respectively.

How do pedestrians respond to vehicles when preparing to cross or crossing the street? Just over half (58%) of the pedestrian crossings that occurred in the presence of vehicles were considered to be normal crossings, implying no unusual behaviors on the part of the pedestrian. The non-normal behavior that was observed most often was hesitation, which could occur on the entry-side or exit-side curb before starting to cross or on the splitter island when preparing to complete the crossing. Most often, the hesitation occurred while the pedestrian made visual or other contact with the driver. Once this communication was made and the vehicle began slowing, the pedestrian would then proceed with the crossing. Across all sites, the average number of crossing events in which a pedestrian hesitated on the curb or after starting was 28%. There was a difference in the percentage of hesitation crossings associated with the number of lanes at the crosswalk. When averaged across both sides of the crossing and from both starting positions, single-lane sites resulted in hesitation crossings 24% of the time, while two-lane sites produced such crossings 33% of the time.

When averaged across both sides of the crossing, approximately 22% of the pedestrians that started crossing from the entry side hesitated, compared to 31% of those that started from the exit side. Irrespective of the starting location, the majority of hesitations occurred at the curb prior to initiating the crossing, rather than halfway through the crossing while on the splitter island. For example, pedestrians hesitated 33% of the time on the entry-side curb when starting from the entry side and only 10% of the time at the splitter island. Similarly, pedestrians starting from the exit-side curb hesitated 39% of the time at the exit-side curb and 23% at the splitter island.

The other crossing behavior that was observed quite often was running. The running behavior in this case was not a "sudden" behavior on the part of the pedestrian to avoid a conflict (*see next question*). It appears to have been simply a choice made by the pedestrian to speed up the crossing and was most often done as a courtesy to the yielding motorist. The average number of pedestrian crossings across all sites in which running was an observed behavior was 14%. For one-lane sites, 17% of the crossings involved a running pedestrian, compared to 8% of the crossings at two-lane sites.

Running was most prevalent for crossings that began on the entry side (18%) as opposed to the exit side (7%). The running behavior was most often observed on the second half of the crossing. For example, of the pedestrians starting on the entry side, 7% ran across the entry side, and 29% ran across the exit side. The same was true for exit-side starts, but less pronounced; 3% ran across the exit side, while 10% ran across the entry side.

Did the behaviors of motorists and pedestrians create unsafe situations? The measure of safety that is most often applied to the roadway environment is a crash. There were no crashes in the pedestrian-vehicle interactions observed in this study. As a surrogate, conflicts between the two modes that required one or both parties to suddenly change course and/or speed to avoid a crash were studied. Out of 769 pedestrian crossings across the 10 sites, there were only four conflicts. The resulting conflict rate was 2.3 conflicts/1000 opportunities. An opportunity was defined as any time a pedestrian was either waiting to cross or crossing the leg *and* a motor vehicle was in the vicinity of the pedestrian. This rate was slightly greater for one-lane sites (2.8) than for two-lane sites (2.0).

What are the geometric or operational characteristics that tend to cause problems for pedestrians or tend to result in safer and more accessible designs? Based on the answers to the previous questions, the two design elements that correlate to differences in behaviors are the number of lanes and the directional side of the site (entry lanes versus exit lanes).

More lanes resulted in a higher number of vehicles not yielding to crossing/waiting pedestrians (43% on two-lane sites versus 17% on one-lane sites). Pedestrian behaviors also differed at these two types of sites, but the results were not consistent. Crossings in which the pedestrian hesitated were 33% on two-lane sites compared to 24% on one-lane sites. Crossings in which the pedestrian ran were 8% for two-lane sites compared to 17% for one-lane sites.

The behaviors of both motorists and pedestrians also differed depending on the directional side of the site: entry versus exit. Motorists were less likely to yield to pedestrians on the exit side (38% of the time, based on an average of entry-side and exit-side starts from Figures 62 and 63) compared to the entry side (23% of the time, based on a similar average). Pedestrian behaviors on the two sides differed according to the starting position of the crossing. Pedestrians were more likely to hesitate when starting from the exit side (31% of the crossings) than when starting from the entry side (22% of the time). Pedestrians were also more likely to run across the exit side when it was the second stage of the crossing than the entry side when it was the second stage (29% versus 10%, respectively).

Further review of the geometric and operational characteristics associated with individual sites was undertaken to determine if specific elements—such as lane widths, splitter island designs, and other factors—were associated with the behaviors observed. These reviews did not produce any additional insights into the observed differences between locations.

How do the behaviors of pedestrians and motorists at roundabouts compare to the behaviors of pedestrians and motorists at other types of intersections? The pedestrian and motorist behavior results for the roundabout sites in this study were compared to the results for two-way-stop–controlled, all-way-stop–controlled, and signalized intersections obtained from the FHWA study titled "Safety Index for Assessing Pedestrian and Bicyclist Safety at Intersections" (*3*). The FHWA results showed 70% of the crossings at intersections with no traffic control were considered normal (i.e., no running, hesitation, or stopping). The next highest percentage (85%) of normal crossings occurred at roundabouts, which are yield-controlled intersections. The crossings at signalized and stop-controlled intersections exhibited the highest normal crossing percentages at 90% and 100%, respectively. Running was the single pedestrian behavior that was substantially more common at roundabouts than at the other types of controlled intersections. As noted previously, however, the running behavior observed at roundabouts appears to have been most often done as a courtesy on the part of the pedestrian.

Pedestrian waiting times and crossing pace (walking speed) were also compared among the four levels of traffic control. The traffic control type that resulted in the longest pedestrian waiting time of 10.7 s was signalization, which is expected because most pedestrians will comply with the signal. Crossings with no traffic control produced an average waiting time of 3.0 s, while crossings at stop-controlled locations resulted in virtually no waiting time (0.3 s). For roundabouts, the average waiting time was 2.1 s, which is between the time for the stop-controlled locations where pedestrians are confident that the vehicle will stop and the no-control locations where there may be an increase in uncertainty about the intent of the vehicle. With respect to pace, there were no practical differences on the basis of traffic control type; all locations ranged between 4.4 and 5.0 ft/s (1.3 to 1.5 m/s).

With respect to motorist behavior and the type of traffic control, almost half (48%) of the vehicles on uncontrolled legs did not yield to pedestrians. The stop-controlled and signalized intersections produced non-yielding vehicle percentages of 4% and 15%, respectively. Roundabout crossings, which are subject to yield control, were in between these extremes with 32% of the motorists not yielding.

Bicyclist Analysis

Data were collected for 690 bicyclist events at 19 legs distributed among seven roundabouts. Only two of these sites had two lanes, so the analysis did not include any one-lane/two-lane site comparisons. Bicyclists were observed as they entered, exited, or circulated in the roundabout or crossed at the crosswalk. Data were collected for all bicyclists who entered the study area shown in dashed lines in Figure 70. The study area included the part of the circulating lanes near the entry/exit of the leg and the leg as far as the end of the splitter island.

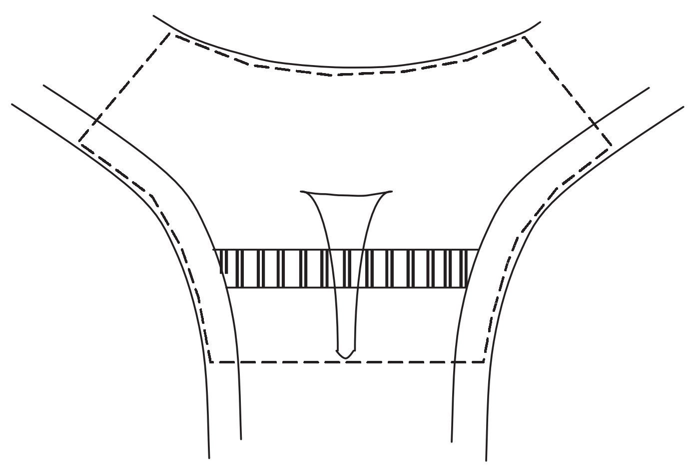

Figure 70. Bicyclist study area.

The analysis of bicyclist events covers the following topics:

- Bicyclist position
- Bicyclist behaviors
- Motorist behaviors
- Bicycle-motor vehicle conflicts
- Other bicyclist behaviors

Refer to Appendix L for images to help describe some of the behaviors observed and described in the subsequent sections.

Bicyclist Position

Bicyclist position refers to the location of the bicyclist's path as the bicyclist enters, exits, or traverses the roundabout. This study classified bicyclist position as on the sidewalk; on the shoulder, bike lane, or edge of travel lane; or possessing the vehicle lane. There were not enough sites with and without bike lanes and paved shoulders to allow for comparison between these configurations. Bicyclist positions classified as edge of lane in the following analyses include positions on bike lanes and paved shoulders. Possessing the lane was defined as a bicyclist riding close to the center of the lane such that motorists would not attempt to pass.

Bicyclists were most commonly found to ride on the edge of the lane. Just over half (54%) rode in this position (see Table 65). The remainder was split between possessing the lane and riding on the sidewalk, with the latter being the least common position.

Position by Event Type

Event type refers to the type of movement that the bicyclist made at the roundabout—entering, exiting, or circulating the roundabout or crossing at the crosswalk. Figure 71 shows that the majority of bicyclists entering or exiting were positioned on the edge of lane, whereas circulating bicyclists more often possessed the lane. A higher percentage of bicyclists entering the roundabout were positioned at the edge of the lane when compared to bicyclists exiting the roundabout (73% entering versus 61% exiting). The most likely explanation for this difference is that entering bicyclists more often had to compete with vehicles for roadway space. Because of queuing at the entry, many entering bicyclists were observed to enter simultaneously with a vehicle or follow very closely behind a vehicle as they entered.

Table 65. Distribution of bicyclists by position.

Position Category	Bicyclists
Edge of Lane/Shoulder/Bike Lane	54%
Possessing Lane	28%
Sidewalk	18%
Total	*100%*

Vehicle Presence and Position of Bicyclists

Bicyclists that ride in the roadway, as opposed to the sidewalk, frequently interact with vehicles and must make decisions about where to position themselves on the basis of factors such as their own comfort level in traffic, the amount of space available, and speed and volume of motor vehicle traffic. For each bicyclist that entered, exited, or circulated in the roundabout, data were collected on the presence and proximity of motor vehicles to the bicyclist. Motor vehicle presence was classified as one of the following:

- Leading bicyclist within two car lengths
- Trailing bicyclist within two car lengths
- Passing bicyclist
- Queued in front of bicyclist (entering bicyclists only)
- Queued behind bicyclist (entering bicyclists only)
- No vehicle leading, trailing, passing, or queued near bicyclist

Out of 450 events in which bicyclists were entering, exiting, or circulating, only 6 events were observed to involve queued vehicle presence. Five bicyclists had vehicles queued in front of them, and one bicyclist had a vehicle queued behind. In all but one case, the bicyclist was positioned on the shoulder. The other categories for vehicle presence occurred more frequently and are shown in Table 66. When there were no vehicles in the vicinity, 42% of bicyclists possessed the vehicle lane. When a vehicle was leading the bicyclist (within two car lengths), the percentage decreased to 35%. When a vehicle was trailing the bicyclist, even fewer bicyclists (23%) possessed the lane. The 12 percentage point difference from leading vehicles to trailing vehicles may indicate that bicyclists were not as comfortable possessing the lane when a vehicle was approaching them from the rear.

The category denoted as "more than one type" indicates that the bicyclist had at least two vehicles in proximity. Most often, the bicyclist had one vehicle leading and another one trailing. As would be expected, the position percentages associated with this occurrence falls in between the values for "leading" and "trailing." In the case of passing vehicles, all but 1 event out of the 37 observed had the bicyclist positioned on the shoulder. In that one event, the bicyclist was on the shoulder for the part of the observation when the passing occurred.

Bicyclist Behaviors

Bicyclist behavior was captured for the two event types where a bicyclist had to accept and enter a gap—entering the

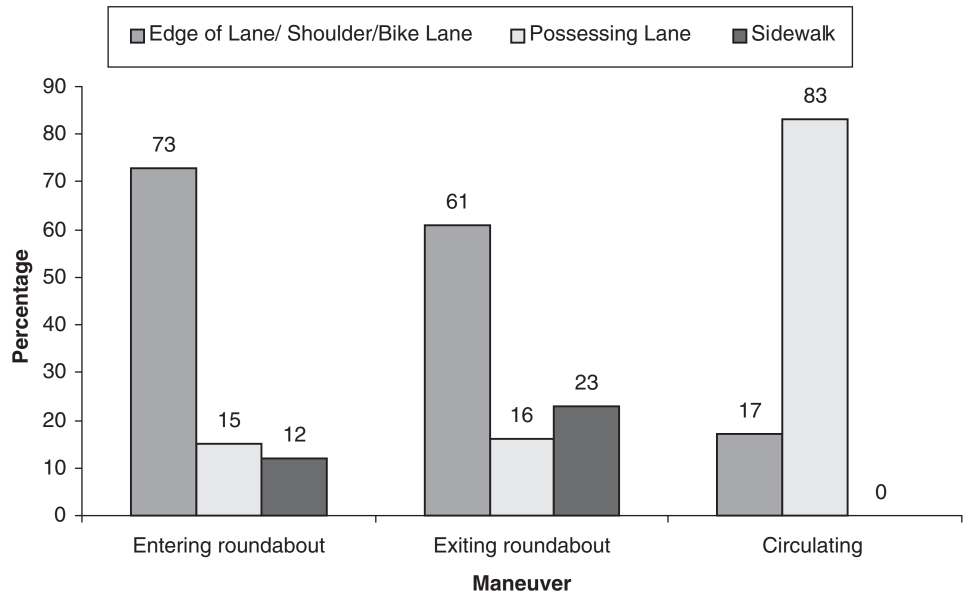

Figure 71. Position of bicyclist on the basis of event type or maneuver at the roundabout.

Table 66. Distribution of bicyclist positions by vehicle presence.

Motor Vehicle Presence	Edge of Lane/ Shoulder/Bike Lane		Possessing Lane		Total	
	Number	%	Number	%	Number	%
None	155	**58%**	113	**42%**	268	**100%**
Leading	31	**65%**	17	**35%**	48	**100%**
Trailing	48	**77%**	14	**23%**	62	**100%**
More than one type	21	**66%**	11	**34%**	32	**100%**
Passing	36	**97%**	1	**3%**	37	**100%**
Total	*291*	***65%***	*156*	***35%***	*447*	***100%***

circulating lane of the roundabout and crossing at the crosswalk. Behaviors were categorized as one of the following:

- Normal (passes through without stopping because there is no vehicle in the vicinity or vehicle yields)
- Hesitates or waits before starting because of approaching vehicle
- Hesitates after starting because of approaching vehicle
- Retreats after starting because of approaching vehicle
- Swerves to avoid approaching vehicle

For each entering or crossing event, the data coding process also included a safety code. Each gap was subjectively deemed to be safe or unsafe, depending on the proximity and speed of nearby vehicles. For example, if a bicyclist approached the crosswalk at a fast speed and crossed without looking at or yielding to oncoming traffic, the event was coded as unsafe. An "unsafe" code could be given even if the interaction did not result in a conflict or crash.

Bicyclist Behaviors on Entering the Roundabout

Out of 238 bicyclists that entered a roundabout, 70% proceeded into the circulating lane without stopping, 11% waited before entering the circulating lane, and 19% entered on the sidewalk. There was one case of a bicyclist swerving to avoid a vehicle. The swerving case was a conflict caused by the bicyclist entering from the exit side and is described in more detail in the conflicts section.

The difference in percentages between proceeding-without-stopping and waiting-before-entering is most likely correlated with the amount of vehicular traffic in the circulating lane, but these data were not available to confirm this hypothesis. From subjective observations, sites with heavier traffic caused bicyclists to yield more frequently to circulating vehicles. Only two cases of entering bicyclists were deemed unsafe. One was the swerving conflict, and the other was a case where the circulating vehicle did not yield but passed on the bicyclist's right. The safe entrance on the roadway or the sidewalk of almost all bicyclists indicates that there are not significant safety problems with bicyclists entering roundabouts.

Bicyclist Behaviors at Crosswalks

There were 81 events where a bicyclist crossed at the crosswalk. Eighty-five percent of these crossings were entirely on the painted crosswalk. Only a few bicyclists rode on the exit-side crosswalk and moved off it for the entry side (7%) or rode on the entry-side crosswalk and moved off it for the exit side (7%).

The observed behaviors included normal (proceeding through without stopping), waiting or hesitating before starting to cross, and hesitating after starting. Table 67 shows that normal behavior was the most common behavior, although there was a difference on the basis of which side was being crossed. More bicyclists waited or hesitated before crossing the exit side (27%) than did at the entry side (20%). Only three bicyclists hesitated after starting—two on the exit side and one on the entry side. Overall, there was more hesitation at the exit side. Not only do vehicles travel faster when exiting the roundabout than when entering, but also bicyclists are uncertain whether a vehicle approaching in the circulating lanes will exit. These conditions may explain the higher bicyclist hesitation rates on the exit side.

Motorist Behaviors

Motorist behavior was recorded when a bicyclist entered the roundabout or crossed at the crosswalk. The motorist behavior was categorized as one of the following:

- **Slows or stops for waiting bicyclist**: Motorist yields to bicyclist waiting on curb or splitter island.
- **Slows or stops for bicyclist in transit:** Motorist yields to bicyclist in motion.
- **Already stopped for other reason:** Motorist is queued at entry or already yielding to other bicyclist or pedestrian and remains stopped to yield to bicyclist.
- **Swerves:** Motorist changes direction to avoid bicyclist.
- **Does not yield:** Motorist does not yield to bicyclist in transit or waiting on curb or splitter.
- **No vehicle present:** No vehicle was in the vicinity at the time of the event.

Table 67. Bicyclist behavior at crosswalks by side.

Bicyclist Behavior	Entry Side		Exit Side	
	Number	%	Number	%
Normal	64	**79%**	57	**70%**
Waits/hesitates before starting	16	**20%**	22	**27%**
Hesitates after starting	1	**1%**	2	**2%**
Total	*81*	***100%***	*81*	***100%***

Motorist Behavior When Bicyclists Entered the Roundabout

There were 238 events involving a bicyclist entering a roundabout. However, 45 of these bicyclists entered on the sidewalk; thus, there were only 193 opportunities for interaction with vehicles. Of these 193 opportunities, 188 (97%) occurred with no vehicle present. That is to say, when the bicyclist accepted the gap and entered the roundabout, there was no vehicle in the circulating lanes to immediately follow the bicycle and "close" the gap. For the remaining five events, a vehicle was present and the following behaviors occurred:

- One motorist slowed for the waiting bicyclist.
- One motorist stopped for the bicyclist entering the roundabout.
- One motorist swerved to avoid the bicyclist.
- One motorist passed the entering bicyclist on the left.
- One motorist passed the entering bicyclist on the right.

The case of the swerving motorist resulted in a conflict and is described in more detail in the section on bicycle-motor vehicle conflicts. The two cases where the motorist passed the entering bicyclist did not result in conflicts but were cases where the bicyclist did not select an appropriate gap.

Overall, there were few opportunities for motorist–bicyclist interaction on entry. For the most part, bicyclists chose to enter the roundabout when there were no vehicles immediately approaching. The selected gaps were large enough that there was no need for approaching vehicles to yield.

Motorist Behavior at Crosswalks

The majority of the 81 crossing events also occurred without a vehicle present. Table 68 shows the distribution of motorist behaviors at the crosswalk, specific to the entry and exit sides of the leg. It was more common that the bicyclist would encounter no vehicles on the exit side (77%) than on the entry side (62%). For the other cases when a vehicle was present, the motorist was always observed to yield to the bicyclist, whether waiting on the curb or in motion on the crosswalk. There were 17 cases where the motorist was already stopped and remained stopped for the bicyclist to pass; all but one of these cases occurred on the entry side.

Table 68. Motorist behavior at crosswalks by side.

Motorist Behavior	Entry Side		Exit Side	
	Number	%	Number	%
No vehicle present	50	**62%**	62	**77%**
Slows or stops for waiting bicyclist	4	**5%**	9	**11%**
Slows or stops for bicyclist in transit	11	**14%**	9	**11%**
Already stopped	16	**20%**	1	**1%**
Total	*81*	***100%***	*81*	***100%***

Unlike the pedestrian study, behaviors of the motorist were only recorded for interactions when there was an accepted gap by the bicyclist. For this reason, Table 68 does not provide information on the percentage of motorists not yielding to bicyclists. For this information, the results found from the pedestrian-motor vehicle interactions, which included a significantly larger number of crossing events at a wider range of geometric conditions, are deemed to suffice.

Bicycle-Motor Vehicle Conflicts

One of the surrogate measures of safety for bicyclists is a conflict with a motor vehicle. In this study, as well as many others, a conflict was defined as an interaction between a bicyclist and motorist in which one of the parties had to suddenly change course and/or speed to avoid a crash. During the 690 bicyclist events, only four conflicts were observed (0.6%). Two of these conflicts occurred while a bicyclist was crossing at the crosswalk; one occurred while a bicyclist was circulating in the roundabout; and one occurred while a bicyclist was entering the roundabout. Each conflict is further described below:

- **Conflict 1 – OR01-N1 (Bend, Oregon) at crosswalk:** Bicyclist begins crossing crosswalk from exit side and continues across entry side without slowing. Vehicle in queue to enter roundabout is in forward motion very close to crosswalk and has to slam on brakes to avoid hitting the bicyclist (Figure 72).
- **Conflict 2 – OR01-S1 (Bend, Oregon) at crosswalk:** Bicyclist begins to cross crosswalk from exit side with little slowing. Vehicle exiting roundabout brakes suddenly to avoid crash (Figure 73).
- **Conflict 3 – WA03-E3 (Bainbridge Island, Washington) circulating:** Bicyclist circulating roundabout on the outside of the circulatory roadway attempts to continue circulating past the leg. A vehicle to the left of the bicyclist attempts to exit the roundabout. Both parties swerve and brake suddenly (Figure 74).
- **Conflict 4 – WA03-S2 (Bainbridge Island, Washington) entering:** Bicyclist approaches roundabout going the wrong way on the exit lane. He begins to enter the roundabout from the exit lane and swerves to avoid an exiting vehicle. Exiting vehicle also swerves to avoid crash. Bicyclist appears to be young (Figure 75).

Figure 72. Bicycle Conflict 1 at OR01-N (Bend, OR).

Figure 73. Bicycle Conflict 2 at OR01-S (Bend, OR).

Other Bicyclist Behaviors

Events involving bicyclist-pedestrian interaction or wrong-way riding are less common behaviors but were encountered in the course of the data collection. There were three cases

Figure 74. Bicycle Conflict 3 at WA03-E (Bainbridge Island, WA).

Figure 75. Bicycle Conflict 4 at WA03-S (Bainbridge Island, WA).

where a bicyclist interacted with a pedestrian. Each interaction occurred when the bicyclist was on the roadway and yielded to a pedestrian on the crosswalk. Nothing significant resulted from these interactions.

Wrong-way riding was defined as a bicyclist riding on the paved roadway contrary to traffic flow. If the bicyclist was on the sidewalk, the event was not coded as wrong-way, even if the motion of the bicyclist was contrary to the flow of the roundabout (i.e., bicyclist entering the roundabout on the exit-side sidewalk). Seven cases of wrong-way riding were recorded. All seven involved a bicyclist entering the roundabout from the exit lane. Of the seven cases, one resulted in a conflict (see Conflict 4 in the previous section). Another point to consider is that five of the wrong-way cases occurred at one roundabout where the camera view did not allow for observation of the bicyclists' final positions once they passed through the study area and appeared to enter the roundabout. It is possible that these bicyclists proceeded to get on the sidewalk to enter the roundabout. Overall, wrong-way riding was a rare event and only once resulted in a safety problem.

Analysis of Findings

How do bicyclists and motorists interact on the entry lanes, exit lanes, and circulating lanes of the roundabout? Where does the bicyclist tend to be positioned? The majority (73%) of bicyclists approaching a roundabout positioned themselves at the edge of the travel lane or on a bike lane or paved shoulder if available. Only 15% of the approaching bicyclists possessed the lane. The remaining 12% used the sidewalk. For exiting bicyclists, 23% used the sidewalk, while 16% possessed the lane. Those bicyclists in the circulating lane tended to take the lane (83%) rather than ride on the edge of the circle.

With respect to other types of interactions between the two modes, no problems were observed. Bicyclists that entered the circulating roadway from the entry lane almost always selected gaps in which no vehicle was approaching on the circulating roadway. Bicyclists and motorists traversed the circulating lane with very little interaction.

Are there conflicts or avoidance maneuvers due to the interactions of motorists and bicyclists? As a surrogate measure for safety, a conflict was defined in this study as an interaction between a bicyclist and motorist in which one of the parties had to suddenly change course and/or speed to avoid a crash. Only four conflicts were observed during the 690 bicyclist events, or 0.6%. Two of these conflicts occurred while a bicyclist was crossing at the crosswalk; one occurred while a bicyclist was circulating in the roundabout; and one occurred while a bicyclist was entering the roundabout. The one involving the bicyclist circulating the roundabout involved an exiting vehicle that almost struck a bicyclist who was traversing the circulating lane on the outside of the lane, which is one of the most vulnerable positions to be in as a bicyclist.

Do bicyclists exhibit any behaviors that raise safety concerns? The one behavior observed that did raise some concern was wrong-way riding, particularly when entering the roundabout from the exit lane of the leg. This scenario did result in one of the four conflicts observed. While the number of observed wrong-way events was small (seven), this type of event can produce crashes as a result of expectancy violations.

Other Design Findings

This portion of the design analysis contains the evaluation of the safety and capacity modeling efforts and their respective observations as they relate to specific design elements of the roundabout. The general approach to this analysis is twofold:

- The sensitivity of various geometric parameters was tested in the development of prediction models for safety and capacity. These tests are documented in the model development for safety and capacity, respectively.
- Sites with abnormal safety and/or capacity performance were examined using expert judgment to identify geometric elements that could be contributing factors to the abnormal safety and/or capacity performance. This analysis was not conducted to the same level of statistical rigor as the analysis associated with model development, but it contributes to an anecdotal understanding of the geometric elements that appear to have an influence on safety and capacity. This anecdotal understanding is presented here.

The safety and geometric data identify several trends related to the early roundabout experience in the United States. Overall, the crash experience has been positive (showed an overall reduction in crash frequency); however, there were several intersections where this was not the case. In some cases, either there was no change in crash frequency or there was actually an increase; although, in almost all cases, the crash counts are too small for the increase to be statistically significant. Many of the roundabouts in the dataset were constructed before the publication of the FHWA Roundabout Guide (*1*).

In general, the evaluation presented here focuses on crash frequency as the primary measure for flagging sites with high or low crash experience. However, this evaluation also compares sites sorted by crash frequency to the sites sorted by crash rate. This analysis confirmed the conclusion that multilane roundabouts represent a greater risk for crashes than single-lane roundabouts.

Table 69 presents an analysis of the relationship of overall roundabout geometry to crash frequency; Table 70 presents a similar analysis of overall roundabout geometry to crash rates. The analysis compares the roundabout geometry across a range of crash frequencies and crash rates: the full dataset, the 10 lowest, the 30 lowest, the 30 highest, and the 10 highest. From this analysis, the following conclusions related to the number of lanes in the roundabout were reached:

- The site sorting from best to worst generally stayed the same whether sorting by crash frequency or crash rate. In general, the use of crash rates for comparisons is not preferred because of the known non-linear relationship between traffic volume and crash frequency. Therefore, the remaining analysis has been conducted using crash frequency.
- Eight of the ten sites with the lowest crash frequencies were single-lane roundabouts.
- Twenty-six of the thirty sites with the lowest crash frequencies were single-lane roundabouts.
- Two of the ten sites with the highest crash frequencies were single-lane roundabouts.
- Nine of the thirty sites with the highest crash frequencies were single-lane roundabouts.
- Crash frequency increases as the inscribed circle diameter increases.
- Crash frequency increases as the amount of vehicles entering the roundabout increases.
- Crash frequency increases slightly as the number of legs to the roundabout increases.

Single-Lane Roundabout Evaluation

Review of the plans for these sites clearly indicates that a designer can get by with making more design errors with single-lane roundabouts. Not all of the single-lane roundabout designs were "perfect," but the designs contained enough geometric changes to indicate the change in intersection form to the drivers, to slow the drivers, and therefore to increase the safety of the intersection. Many of the single-lane roundabouts were also clustered in several states, so driver familiarity may have played a role. For the single-lane roundabouts that did result in a higher crash frequency, there was little deflection or speed reduction on the entry paths to the roundabout. For example, the single-lane roundabout with the highest crash frequency in the dataset is shown in Figure 76. The speeds estimated from fastest paths (using the current FHWA Roundabout Guide methodology) exceed the thresholds recommended in the FHWA Roundabout Guide. There is little or no deflection on the approaches to the

Table 69. Relationship between crashes and geometry, sorted on crash frequency.

	Crash Frequency (crashes/yr)	Crash Rate (crashes/MEV)	Average Number of Lanes in Group	Average Inscribed Circle Diameter	Average Daily Traffic (veh/day)	Average Number of Legs in Group
Total Dataset	4.95	0.75	1.39	133 ft (41 m)	16,606	3.89
First Ten	0.01	0.00	1.20	97 ft (30 m)	8,604	3.60
First Thirty	0.44	0.16	1.13	114 ft (35 m)	9,585	3.67
Bottom Thirty	12.13	1.59	1.83	162 ft (49 m)	23,935	4.13
Bottom Ten	22.89	2.64	2.20	215 ft (66 m)	28,300	4.30

Legend: MEV = million entering vehicles; veh = vehicles

Table 70. Relationship between crashes and geometry, sorted on crash rates.

	Crash Frequency (crashes/yr)	Crash Rate (crashes/MEV)	Average Number of Lanes in Group	Average Inscribed Circle Diameter	Average Daily Traffic (veh/day)	Average Number of Legs in Group
Total Dataset	4.95	0.75	1.39	133 ft (41 m)	16,606	3.89
First Ten	0.02	0.00	1.20	95 ft (29 m)	9,295	3.70
First Thirty	0.59	0.10	1.23	123 ft (37 m)	14,961	3.73
Bottom Thirty	11.75	1.69	1.70	165 ft (50 m)	20,186	4.07
Bottom Ten	18.51	3.03	1.90	150 ft (46 m)	16,734	4.20

Legend: MEV = million entering vehicles; veh = vehicles

roundabouts, and the research team believes that this is the primary cause for the high crash frequency.

Multilane Roundabout Evaluation

Approximately one-third of the sites in the safety database are multilane roundabouts. However, 8 of the 10 sites with the highest crash frequency were multilane roundabouts. Therefore, it is apparent that at least some multilane roundabouts have abnormally high crash experiences that warrant further investigation.

Review of these sites led the research team to believe that most were not designed using the natural vehicle path concept. This scenario is entirely likely because the majority of these sites were designed and constructed before the publication of the FHWA Roundabout Guide (*1*), which was the first document to publish this concept. The natural vehicle path concept, shown in Figure 77, was refined in later guidelines such as the Kansas Department of Transportation's *Kansas Roundabout Guide* (*37*).

Lane widths also appear to have an effect on safety. For example, one roundabout appeared to be designed to

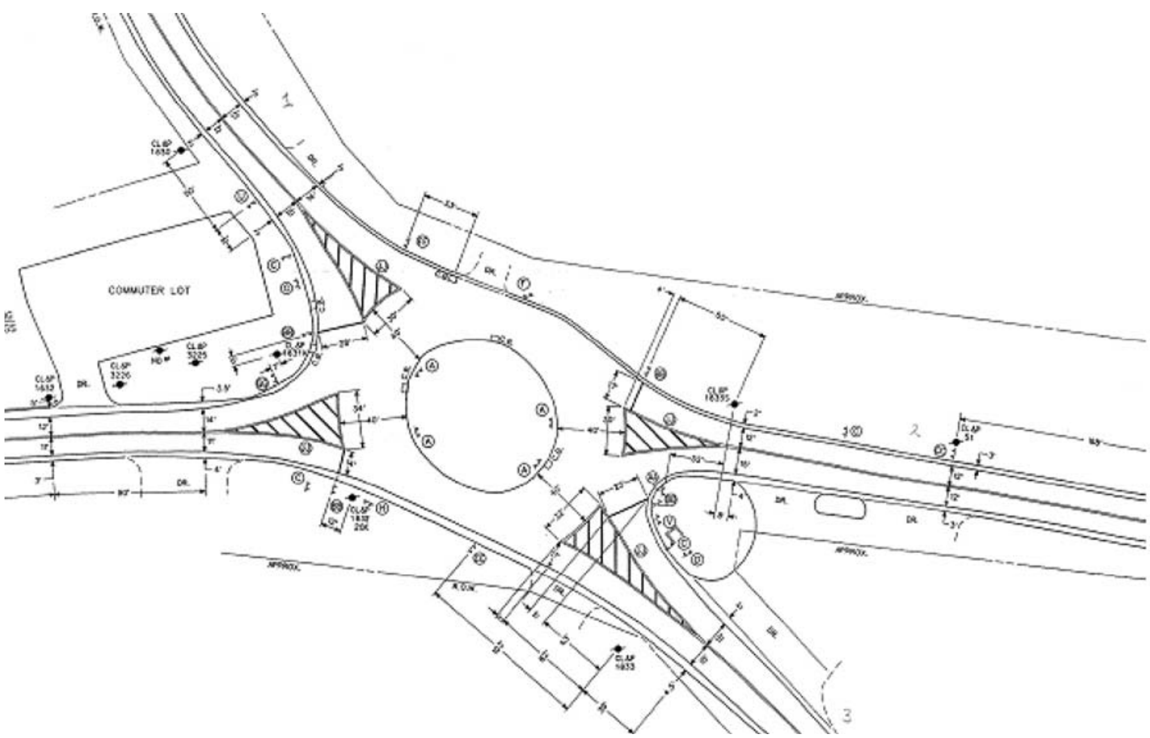

Figure 76. Example of a single-lane roundabout with poor deflection characteristics.

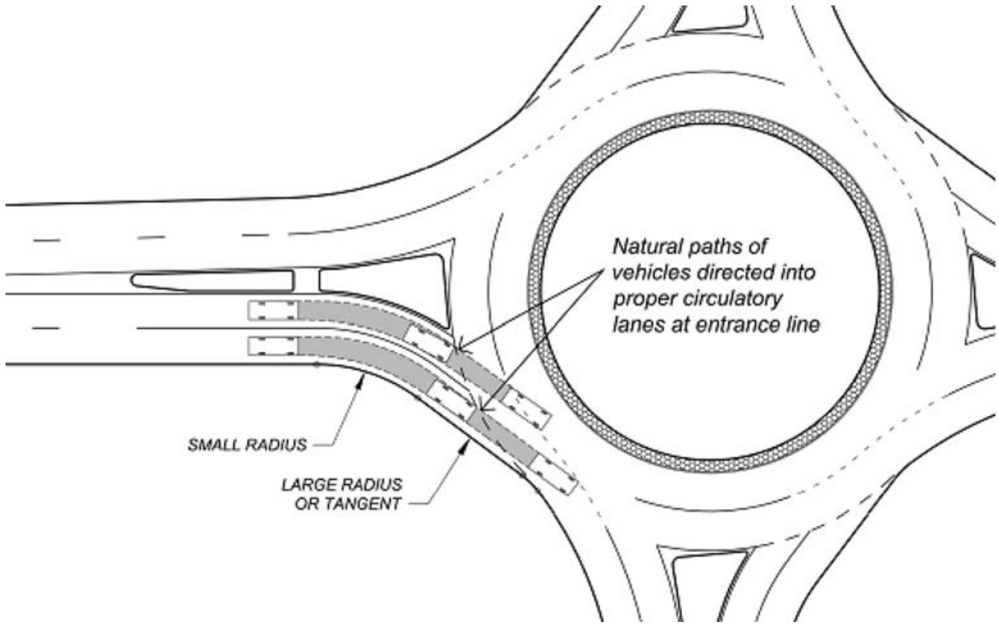

Figure 77. Design technique to minimize entry path overlap.

accommodate the natural vehicle path yet still exhibited a higher than anticipated crash frequency. This site exhibits narrower lane widths than other sites reviewed and than recommended by the FHWA Roundabout Guide. The intersection geometry for the site is shown in Figure 78. The FHWA and other guidebooks recommend lane widths in the range of 13 to 16 ft (4.0 to 4.9 m) at the entries and exits to the roundabouts, and circulatory roadway widths of 16 to 20 ft (4.9 to 6.1 m) for single-lane roundabouts and 26 to 30 ft (7.9 to 9.1 m) for multilane roundabouts. For this particular site, the entry lane widths of 10 to 11 ft (3.0 to 3.4 m) were maintained on the approaches to the yield line. The circulatory roadway width was also approximately 22 ft (6.7 m) for two circulatory lanes.

The natural vehicle paths and the FHWA speed paths for this site are illustrated in Figures 79 and 80, respectively. Note that the speed paths stay in their respective lanes on the legs; a speed path using the UK method (curb to curb without consideration of lanes) would result in slightly higher predicted speeds. Based on this analysis, the natural vehicle paths and the speed paths through the roundabout did not reveal major problems that would suggest a crash problem based on conventional design wisdom. Therefore, the entry lane widths are potentially the prime design feature contributing to the observed crash experience.

Summary of Design Findings

Overall, the data suggest that roundabouts can improve the safety performance at intersections. However, the research team believes that the performance of the roundabouts discussed here could be improved by relying on the guidance put forth in the FHWA guide and state supplements.

Entry Width

The conventional wisdom in roundabout design, as in intersection design in general, is that as the width of an entry increases, the capacity of the entry increases, while the safety of the entry decreases. In most countries, the safety and operational effect of entry width is related primarily to the number of lanes provided by the entry in question, with wide entries typically having more lanes than narrow entries. Using linear regression, Maycock and Hall established an empirical relationship in the UK between entry width and entering-circulating crashes (*4*); this model uses entry width as a direct input, rather than the number of lanes on the entry. Likewise, Kimber determined an empirical relationship between entry width and capacity, also using entry width as a direct input rather than the number of lanes (*19*). Most other known safety and capacity models are based on the number of lanes rather than the actual entry width.

Analysis of U.S. data suggests that this principle generally holds true for U.S. conditions. Entry width was found to have a direct relationship with entering-circulating crashes and is part of the candidate model for estimating such crashes (see safety analysis for more detail). In addition, entry width in the aggregate sense—number of lanes—appears to have a direct relationship on capacity, as evidenced by the development of single-lane and multilane capacity models.

However, the extension of the principle beyond number of entry lanes to the actual width of the entry does not appear to

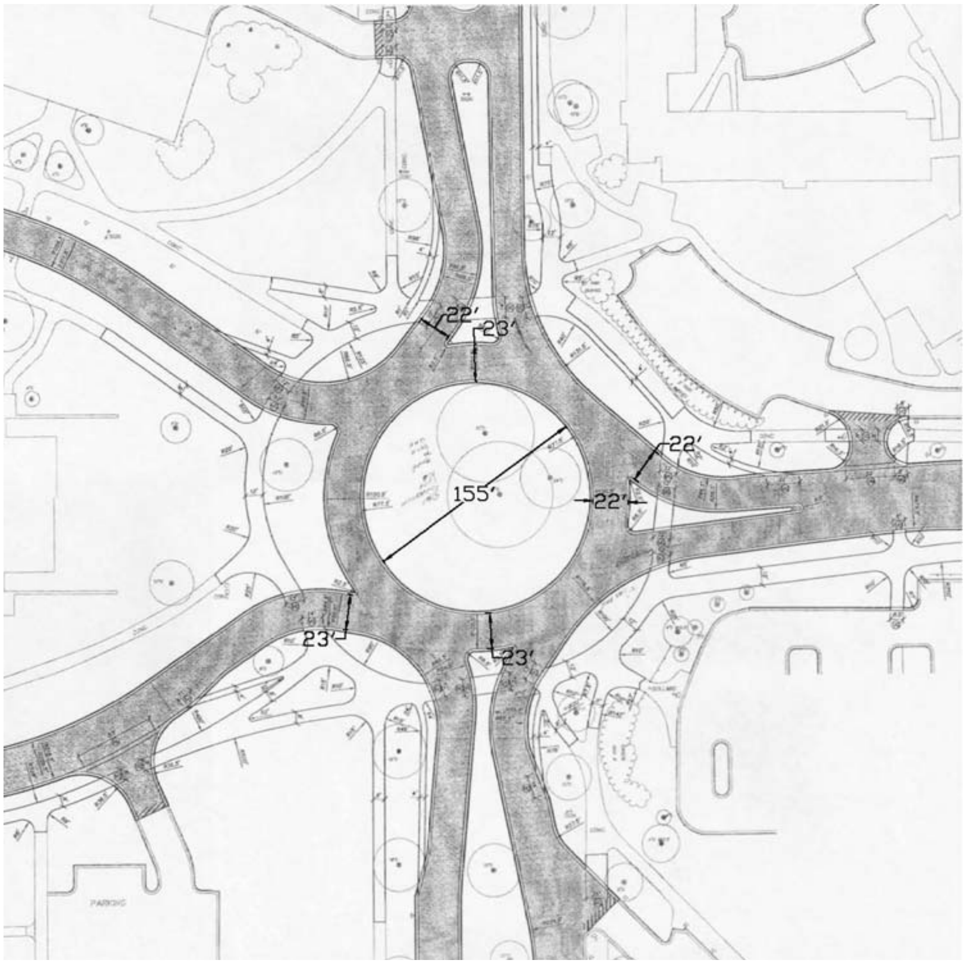

Figure 78. Example of a multilane roundabout with narrow entry and circulatory roadway widths.

have as strong a relationship in the United States. As demonstrated in the operational analysis work, while there appears to be a relationship between the additional width added as part of entry flare (see Chapter 4), there appears to be no significant effect on capacity for variations of entry width within a single-lane entry. This evidence suggests that, while the overall relationship between capacity and entry width appears to hold true in terms of the aggregate number of lanes on the approach, changes in entry width within a single-lane entry has a much lower-order effect on capacity. The number of sites with multilane entries is too limited to make similar conclusions about the influence of small changes in entry width on the capacity of multilane entries.

Angle Between Legs

The angle between legs of a roundabout appears to have a direct influence on entering-circulating crashes and is part of the candidate model for estimating such crashes (see Chapter 3 for more detail). As the angle to the next leg decreases, the number of entering-circulating crashes increases. This result is consistent with the experience in the UK, whose entering-circulating crash model also includes this geometric parameter.

From a design perspective, this evidence suggests that roundabouts with more than four legs or with skewed approaches tend to have more entering-circulating crashes. In many of these cases the higher speeds enabled by these designs may be contributing to these higher crash frequencies. An example of this was discussed earlier ("Single-Lane Roundabout Evaluation").

Splitter Island Width and Effect of Exiting Vehicles

The width of the splitter island and its effect on safety and capacity was investigated, because other countries found a relationship. Wider splitter islands were speculated to result in improved entry capacity due to drivers being more able to differentiate between circulating and exiting vehicles. The analysis of U.S. data, however, did not find a significant relationship between the capacity of the entry and the width of the splitter island, nor with the percentage

Figure 79. Natural vehicle paths in a multilane roundabout with narrow entry and circulatory roadway widths.

of exiting vehicles. As a result, this factor has not been included in the recommended capacity models. As noted in the operational analysis, U.S. drivers appear to be navigating roundabouts very cautiously at the present time. The research team believes that as drivers become more comfortable and efficient, the effect of the width of the splitter island and/or percentage of exiting vehicles may become more noticeable and should be studied in the future.

Intersection Sight Distance

The FHWA Roundabout Guide (*1*) presents an intersection sight distance methodology based on critical headway (critical gap) values; this methodology is consistent with the AASHTO Policy (*33*). The FHWA Roundabout Guide recommends a critical headway value of 6.5 s, which was based on an adaptation of the AASHTO Policy values for yield-controlled intersections.

As noted in the operational analysis, drivers exhibit a range of critical headway values based on the type of roundabout (single lane versus multilane). Table 71 summarizes the critical headway analysis conducted for this project for single-lane and multilane approaches for those observations where a driver accepts a gap after rejecting a gap (Method 2, as described in Chapter 4). Note that all of these observations have been made with the waiting driver positioned at the yield line (entrance line).

It is reasonable for the value of critical headway that is used for intersection sight distance calculations to be more conservative than that used for capacity estimation; this philosophy is the same that was employed in revising the intersection sight distance methodology in the 2001 AASHTO Policy, as documented in *NCHRP Report 383* (*38*). Based on these findings, the critical headway estimate of 6.5 s in the FHWA Roundabout Guide appears to be somewhat conservative for design purposes for both single-lane and multilane entries. A lower value of 6.2 s is recommended for design purposes, which represents approximately one standard deviation above the mean observed critical headway. For comparison purposes, the critical headway value recommended by AASHTO for minor-street right turns at a stop-controlled intersection is 7.5 s, which is

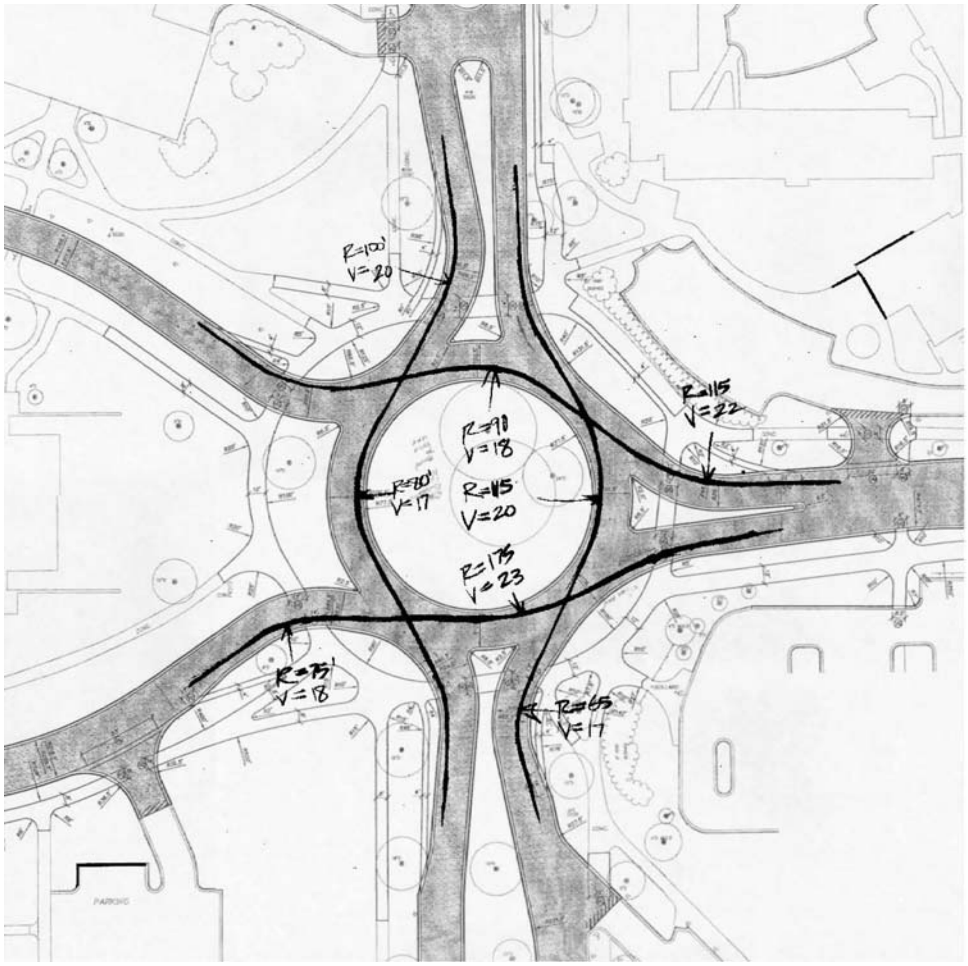

Figure 80. FHWA speed paths in a multilane roundabout with narrow entry and circulatory roadway widths.

greater than the 6.2 s value measured by Kyte et al. (*39*) as part of the procedure to estimate capacity for two-way-stop–controlled intersections (*38*).

Although this study does not recommend a major change in the critical headway estimate, this study has previously presented changes to the methodology for estimating vehicle speeds through a roundabout. These changes influence intersection sight distance, as they dictate the distance over which conflicting vehicles will travel during the elapsed headway time. While the proposed estimation method for circulating vehicles remains unchanged from current practice, the speed estimates for vehicles approaching from the immediate upstream entry are likely to be lower with this revised methodology. In particular, the actual speed estimates for V_1, which forms half of the estimated speed for the intersection sight distance methodology, may be substantially lower than previously estimated using the methodology from the FHWA Roundabout Guide. This lower speed may result in shorter sight triangles to the left (toward the immediate upstream entry); these sight distances are often the most challenging to provide in practice.

Table 71. Critical headway summaries for intersection sight distance.

	Number of Observations	Mean	Standard Deviation
Single Lane	3,322	5.0	1.2
Multilane (both lanes)	3,350	4.5	1.6

The issue of the balance between providing adequate sight distance and providing too much sight distance has not been addressed in this study. As a result, the recommendations from this study should be viewed as interim until a more comprehensive study of sight distance requirements at roundabouts can be completed.

Multilane Entry and Exit Design

The concept of natural path overlap at multilane roundabouts was first introduced into a design guide with the

FHWA Roundabout Guide in 2000 (*1*) and continued in a number of state guides (e.g., Kansas [*37*]).

The general concept is that for optimal safety and operational performance of roundabout entries and exits, the entry lanes at the entrance line to the roundabout should align with their receiving lanes within the circulatory roadway; likewise, the exit lanes should align with their feeding lanes within the circulatory roadway. The most common case where entry lanes do not line up with their receiving lanes in the circulatory roadway is one where the outermost entry lane lines up with the inside portion of the circulatory roadway. The inner entry lane then lines up with the central island. In these cases, the natural tendency (the natural path) for vehicles in the outer lane is to pass close to the central island, thus effectively impeding vehicles in the adjacent (inner) lane. From a safety perspective, this situation could result in sideswipe crashes; from an operational perspective, this situation could result in poor lane utilization and thus reduced effective capacity.

On the exit side, path overlap can occur where the exit path radius is small relative to the path radius of the circulatory roadway. As vehicles traverse the circulatory roadway in the inside lane, their natural tendency is to proceed along a path of similar radius (and thus similar speed) through the exit. A small exit path radius can cause exiting vehicles in the inner lane to overlap with exiting vehicles in the outer lane. As noted in the safety analysis, the inscribed circle diameter and the width of the circulatory roadway appear to have a direct relationship with exiting-circulating crashes. As both parameters increase in value, the number of exiting-circulating crashes increases. This result is to be expected, as multilane roundabouts are the most likely to experience these types of crashes.

A general analysis of the safety data for the multilane roundabouts within the database suggests that, of the sites with the highest crash frequencies and/or crash rates, the majority exhibit some form of path overlap on entries and/or exits. Likewise, sites without noticeable path overlap tend to have lower frequencies and crash rates.

Anecdotal evidence suggests that corrections to path overlap problems can have noticeable effects on safety performance. These corrections can be geometric changes to entry and exit curvature, striping changes to better define lane positioning, or a combination of the two. Striping changes that have been anecdotally found to be successful consist of exit striping patterns that guide circulating vehicles to the proper exit without the need to change lanes within the circulatory roadway. In January 2006, the National Committee on Uniform Traffic Control Devices approved striping recommendations consistent with this philosophy.

After the Gateway Roundabout in Clearwater Beach, Florida, experienced an unacceptably high number of crashes, it was modified through a combination of geometric and striping changes to reduce the number of these crashes caused by path overlap (*40*). Anecdotal evidence suggests a large reduction in the number of crashes (*41*). While the research team does not dispute the overall improvement made in vehicular crash experience with the implemented changes, it notes that the vast majority of crashes in the before period consisted of "subreportable" PDO crashes for which a special reporting effort was made (*42*); whether these crashes were similarly reported after the improvements were made cannot be confirmed. Therefore, the overall magnitude of improvement to the intersection appears to be substantial but cannot be accurately quantified.

Conclusion

In general, the majority of the roundabouts in the United States appear to operate without any significant operational or reported safety deficiencies. However, the findings from this project suggest a number of areas where special attention is needed to ensure the safe and efficient operation of the roundabout for all users. These areas include the following:

- Multilane roundabouts need to be carefully designed to avoid entry and exit path overlap. The majority of the multilane roundabouts with high crash frequencies and high crash rates relative to the other sites in the database exhibited some degree of vehicle path overlap. In addition, some of these sites also experienced reduced operational performance in terms of unbalanced lane utilization on the approach.
- Roundabout exits tend to have a higher percentage of vehicles that do not yield to pedestrians than roundabout entries. As a result, the design of the exit should be carefully considered to ensure that vehicle speeds are reasonable and that good sight lines exist between drivers and pedestrians. The recommended speed methodologies presented in this report may be used to estimate exit speeds based on the configuration of the roundabout.
- Multilane roundabouts tend to have a higher percentage of vehicles that do not yield to pedestrians on either entry or exit. While no quantifiable crash experience has resulted from this behavior, it may reduce the usability of the roundabout crosswalk for pedestrians.

CHAPTER 6

Interpretation, Appraisal, and Applications

This chapter presents a series of interpretations, appraisals, and applications of the major findings of this study. This chapter is organized into the following sections:

- Application of intersection-level safety performance models
- Estimation of the safety benefit of a contemplated conversion of an existing intersection to a roundabout
- Application of approach-level safety models
- Incorporation of safety models into other documents
- Application of operational models

Application of Intersection-Level Safety Performance Models

The safety models and results presented in Chapter 3 can be used in a number of ways (with appropriate cautions). The intersection-level models can be used to evaluate the safety performance of an existing roundabout and to aid in the estimation of the expected safety changes if a roundabout is contemplated for construction at an existing conventional intersection. The approach-level models are presented as tools for evaluating designs in two optional cases: (1) in direct application or (2) as described in Chapter 3, the models with AADT only can be considered as base models (consistent with anticipated HSM procedures) and allow for the estimated coefficients for geometric features in recommended and other models to be considered in developing CMFs. *At both the intersection and approach levels, the potential user should confirm that the models adequately represent the jurisdiction or can be recalibrated using data from the jurisdiction.* Details of these applications follow, along with a discussion on how these potential applications might be implemented.

The intersection-level models presented in Chapter 3 can be used in an empirical Bayes (EB) procedure to estimate the expected safety performance of an existing roundabout, *providing the models can be assumed as* representative *of the pertinent jurisdiction or can be recalibrated using representative data from that jurisdiction*. This result can then be used in a network screening process to examine the performance of that roundabout in relation to other roundabouts or other intersections. For roundabouts performing below par from a safety perspective, diagnostic procedures can then be used to isolate any problems and to develop corrective measures.

The EB method provides a procedure to combine model predictions and observed crash frequencies into a single estimate of the expected crash frequency, so that the observed crash history of a site can be considered in the estimation process, while recognizing that the observed crash frequency by itself is a poor estimate of expected crash frequency because of the randomness of crash counts.

Overview of EB Calculations

Step 1

Assemble data including the number of legs, the number of circulating lanes, and the count of total and injury crashes (excludes possible injury) for the roundabout of interest for a period of n (up to 10) years. For the same time period, obtain or estimate a total entering AADT representative of that time period.

Step 2

Assuming the model is representative of the jurisdiction, select the appropriate intersection-level model from Table 19 or 20 and then use it to estimate the annual number of crashes that would be expected at roundabouts with traffic volumes and other characteristics similar to the one being evaluated.

If the model cannot be assumed to be representative of the jurisdiction, recalibrate it using data (similar to data acquired in Step 1) from a sample of roundabouts representative of that jurisdiction. At a minimum, data for at least 10 roundabouts with at least 60 crashes are needed. The recalibration multiplier is the sum of crashes recorded in the jurisdiction calibration dataset divided by the sum of the crashes predicted by the model for the jurisdiction calibration dataset. Then use the model from Table 19 or 20 including the recalibration multiplier to estimate the annual number of crashes, P.

Step 3

Combine the SPF estimate, P, with the count of crashes, x, in the n years of observed data to obtain an estimate of the expected *annual* number of crashes, m, at the roundabout. This estimate of m is calculated as

$$m = w_1 x + w_2 P$$

where the weights w_1 and w_2 are estimated from the mean and variance of the model estimate as

$$w_1 = \frac{P}{\frac{1}{k} + nP}$$

$$w_2 = \frac{\frac{1}{k}}{\frac{1}{k} + nP}$$

where k is the dispersion parameter for a given model and is estimated from the SPF calibration process with the use of a maximum likelihood procedure. The same procedure is used with the appropriate models for total and injury crashes.

Example 1

Consider that the calculations for total crashes are of interest for a given roundabout.

Step 1

The assembled data are as follows:

- Number of legs = 4
- Number of circulating lanes = 1
- Years of observed data = n = 3
- Total crashes observed = x = 12
- Total entering AADT = 17,000

Step 2

The appropriate SPF and dispersion factor k from Table 19, given four legs and one circulating lane, are as follows:

$$\text{Total crashes/yr} = 0.0023(\text{AADT})^{0.7490}, \quad k = 0.8986$$

Assume for illustration purposes that this model is representative of intersections in the jurisdiction and that no recalibration is necessary. The estimate of P is then

$$P = 0.0023(17{,}000)^{0.7490} = 3.39 \text{ crashes/yr}$$

Step 3

Calculate the weights and the EB estimate of expected annual crash frequency.

$$w_1 = \frac{P}{\frac{1}{k} + nP} = \frac{3.39}{\frac{1}{0.8986} + 3 \times 3.39} = 0.30$$

$$w_2 = \frac{\frac{1}{k}}{\frac{1}{k} + nP} = \frac{\frac{1}{0.8986}}{\frac{1}{0.8986} + 3 \times 3.39} = 0.10$$

$$m = w_1 x + w_2 P = 0.30 \times 12 + 0.10 \times 3.39 = 3.94 \text{ crashes/yr}$$

Therefore, the prediction model estimate of 3.39 has been refined to an EB estimate of 3.94 after consideration of the observed annual crash frequency of 12 crashes in 3 years.

Application to Network Screening

Part IV of the *Highway Safety Manual* will provide procedures for network screening. It is anticipated that these will be based on EB estimates since this is currently the state of the art. In screening, EB estimates can be used to assess how well an existing roundabout is performing relative to similar roundabouts and other intersection types. Comparisons may be made to the average expected crash frequency of other sites or to specific sites in particular. If the other sites are also roundabouts, the appropriate models would be selected from Table 19. If the other sites are other intersection types, then similar models specific to those site types need to be assembled.

Comparison to the Average Expected Crash Frequency

Comparing the expected crash frequency of a particular roundabout to the average expected frequency involves comparing that site's EB estimate to the regression model estimate for that average site type.

Comparison to Other Specific Sites

This comparison involves comparing the site's EB estimate to the EB estimate for the other sites. A useful application of these estimates is to rank sites in descending order of expected crash frequency to prioritize the sites for a more detailed investigation of safety performance. An alternative method is to rank sites by the difference between the EB estimate and the prediction model estimate. The models and results in Chapter 3 allow for either method to be applied.

Estimation of the Safety Benefit of a Contemplated Conversion of an Existing Intersection to a Roundabout

To provide designers and planners with a tool to estimate the change in crash frequency expected with the conversion of an intersection to a roundabout, two alternative approaches are proposed: calibrated intersection-level models or before-after studies. For both approaches, an SPF representative of the existing intersection is required; that is, an SPF must exist for the jurisdiction or data must be available to enable recalibration of a model calibrated for another jurisdiction. The SPF of the existing intersection would be used, along with the intersection's crash history, in the EB procedure to estimate *the expected crash frequency with the status quo in place (the EB estimate)*, which would then be compared to *the expected frequency should a roundabout be constructed* to estimate the benefit of converting the intersection to a roundabout.

The two approaches differ in how *the expected frequency should a roundabout be constructed* is estimated. For the *preferred* approach (Method 1), this value is estimated from an intersection-level model, which requires that data be available to recalibrate intersection-level models or that existing models be deemed adequate for the jurisdiction. Where there is no representative intersection-level model for the jurisdiction, an *alternative* approach (Method 2) can be used. In this approach, the results of the before-after study presented in Chapter 3 (Table 28) can be applied as CMFs to *the expected crash frequency with the status quo in place* to get the expected benefit.

The first approach (Method 1) is preferred and most convenient because a comprehensive set of CMFs (which would be required for a large number of conditions, including AADT levels) is simply not available and is difficult to obtain, although some have been estimated in the disaggregate analysis conducted for this project.

Overview of the Preferred Approach

For presentation purposes it is assumed that a stop-controlled intersection is being considered for conversion to a roundabout.

Step 1

Assemble data and crash prediction models for stop-controlled intersections and roundabouts. For the past *n* years

a. Obtain the count of total and injury crashes.
b. For the same period, obtain or estimate the average total entering AADTs.
c. Estimate the entering AADTs that would prevail for the period immediately after the roundabout is installed.
d. Assemble required crash prediction models from Chapter 3 or elsewhere for stop-controlled intersections and roundabouts. If the models cannot be assumed to be representative of the jurisdiction, they must be recalibrated using data (similar to data acquired in Step 1a) from a sample of intersections representative of that jurisdiction. At a minimum, data for at least 10 intersections with at least 60 crashes are needed. The recalibration multiplier is simply the number of crashes recorded in the sample divided by the number of crashes predicted for the sample by the model. The multiplier is applied to the equation selected for predicting crashes.

Step 2

Use the EB procedure with the data from Step 1 and the stop-controlled intersection model to estimate the expected annual number of total and injury crashes that would occur without conversion. The EB estimate for PDO crashes is then derived as the EB estimate for total crashes, minus the EB estimate for injury crashes.

Step 3

Use the appropriate intersection-level model from Table 19 or 20 in Chapter 3 and the AADTs from Step 1 to estimate the expected number of total and injury crashes that would occur if the intersection were converted to a roundabout. The estimate for PDO crashes is then derived as the model estimate for total crashes minus the model estimate for injury crashes.

Step 4

Obtain for injury and PDO crashes, the difference between the stop-controlled and roundabout estimates from Steps 2 and 3.

Step 5

Applying suitable severity weights and dollar values for injury and PDO crashes, obtain the estimated net benefit of converting the intersection to a roundabout. The best source of information for unit crash costs at the time of this writing is a recently published FHWA web document, "Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries," at http://www.tfhrc.gov/safety/pubs/05051/index.htm. The document presents disaggregate unit costs of crashes by severity level, by type of facility (e.g., intersection type), by impact type, and by environment (urban versus rural).

Step 6

Compare the net benefit against the cost, considering other impacts if desired and using conventional economic analysis tools. How this analysis is done, and in fact whether it is done, is very jurisdiction specific, and conventional methods of economic analysis can be applied only after estimates of the economic values of changes in delay, fuel consumption, and other impacts have been obtained. The results of this analysis may indicate that roundabout conversion is justified based on a consideration of safety benefits. This result may be considered in context with other factors, such as the following:

- Other improvement measures at the given intersection may have higher priority in terms of cost effectiveness.
- The analysts may need to assess the safety benefits and other benefits (delay, fuel consumption, etc.) against the costs and other impacts that may be created by the roundabout.
- Other locations in a system may be more deserving of a roundabout. In other words, the analyst should feed the results of this analysis into the safety resource allocation process.

Example 2

Consider the data for the roundabout in Example 1. Before it was converted to a single-lane roundabout, this site was a four-leg, two-way-stop–controlled intersection in an urban environment. Assume for purposes of this example that before the roundabout was actually constructed, the proposed process was used to decide whether to convert this site into a roundabout.

Preferred Approach

Step P1. The assembled data are as follows:

- Number of legs = 4
- Control = two-way stop
- Years of observed data = 3
- Total crashes observed = 17
- Injury crashes observed = 10
- Average total entering AADT during years of observed data = 16,000
- Anticipated AADT at time of conversion = 17,000

Step P2. Models for urban, four-leg, two-way-stop–controlled intersections (Table 27) can be used in the EB procedure to predict the expected annual number of crashes if the conversion does not take place.

First, the models are used to predict the annual number of crashes by severity:

$$\begin{aligned}\text{Total crashes/yr} &= \exp(-1.62)(\text{AADT})^{0.220},\ k = 0.45\\ &= \exp(-1.62)(16000)^{0.220}\\ &= 1.66\end{aligned}$$

$$\begin{aligned}\text{Injury crashes/yr} &= \exp(-3.04)(\text{AADT})^{0.220},\ k = 0.45\\ &= \exp(-3.04)(16000)^{0.220}\\ &= 0.40\end{aligned}$$

Next, the weights and EB estimate are calculated for total crashes:

$$w_1 = \frac{P}{\frac{1}{k}+nP} = \frac{1.66}{\frac{1}{0.45}+3\times1.66} = 0.23$$

$$w_2 = \frac{\frac{1}{k}}{\frac{1}{k}+nP} = \frac{\frac{1}{0.45}}{\frac{1}{0.45}+3\times1.66} = 0.31$$

$$m = w_1x + w_2P = 0.23\times17 + 0.31\times1.66 = 4.42 \text{ total crashes/yr}$$

Then, the weights and EB estimate are calculated for injury crashes:

$$w_1 = \frac{P}{\frac{1}{k}+nP} = \frac{1.40}{\frac{1}{0.45}+3\times0.40} = 0.12$$

$$w_2 = \frac{\frac{1}{k}}{\frac{1}{k}+nP} = \frac{\frac{1}{0.45}}{\frac{1}{0.45}+3\times0.40} = 0.65$$

$$m = w_1x + w_2P = 0.12\times10 + 0.65\times1.66 = 2.28 \text{ injury crashes/yr}$$

Because volumes are expected to increase in the after period, albeit only slightly, an adjustment is made to m to account for this change. This factor is calculated as

$$\begin{aligned}(\text{AADT after})^{0.220}/(\text{AADT before})^{0.220} &= (17000)^{0.220}/(16000)^{0.220}\\ &= 1.01\end{aligned}$$

The adjusted m is now equal to

$4.42 \times 1.01 = 4.46$ for total crashes/yr
$2.28 \times 1.01 = 2.30$ for injury crashes/yr

The expected number of annual crashes by severity at the site if a conversion *does not* take place is estimated to be 4.46 total and 2.30 injury crashes/yr. The expected number of annual PDO crashes is calculated as $4.46 - 2.30 = 2.16$.

Step P3. The intersection-level model (see Example 1) is used to predict the annual number of crashes should the intersection be converted. In this case, the model was deemed adequate and was not recalibrated specifically for the jurisdiction.

$$\begin{aligned}\text{Total crashes/yr} &= 0.0023(\text{AADT})^{0.7490}\\ &= 0.0023(17000)^{0.7490}\\ &= 3.39\end{aligned}$$

$$\begin{aligned}\text{Injury crashes/yr} &= 0.0013(\text{AADT})^{0.5923}\\ &= 0.0013(17000)^{0.5923}\\ &= 0.42\end{aligned}$$

The expected number of annual crashes by severity at the site if a conversion does take place is 3.39 total and 0.42 injury crashes/yr. The expected number of annual PDO crashes is calculated as $3.39 - 0.42 = 2.97$.

Step P4. The expected change in total crashes is equal to $3.39 - 4.46 = -1.07$ total crashes/yr, or a 24% reduction.

The expected change in injury crashes is equal to $0.42 - 2.30 = -1.88$ injury crashes/yr, or an 82% reduction.

The expected change in PDO crashes is equal to $2.97 - 2.16 = 0.81$ PDO crashes/yr, or a 38% increase.

Alternative Approach

Step A1. This step is the same as Step P1 in the preferred approach.

Step A2. This step is the same as Step P2 in the preferred approach. The expected number of annual crashes by severity at the site if a conversion does not take place is estimated to be 4.46 total crashes, 2.30 injury crashes, and 2.16 PDO crashes.

Step A3. From Table 28 in Chapter 3, the index of effectiveness for an urban, two-way-stop–controlled intersection converted to a single-lane roundabout is 0.612 for total crashes and 0.217 for injury crashes.

The estimate of crashes per year after conversion is

$4.46 \times 0.612 = 2.73$ total crashes/yr
$2.30 \times 0.217 = 0.50$ injury crashes/yr
$2.73 - 0.50 = 2.23$ PDO crashes/yr

Step A4. The expected change in total crashes is equal to $2.73 - 4.46 = -1.73$ total crashes/yr, or a 39% reduction.

The expected change in injury crashes is equal to $0.50 - 2.30 = -1.80$ injury crashes/yr, or a 78% reduction.

The expected change in PDO accidents is equal to $2.23 - 2.16 = 0.07$ PDO crashes/yr, or a 3% increase.

Difference in Results

The results obtained using the preferred and the alternative approaches differ because the preferred method incorporates data calibrated to the jurisdiction. The alternative approach employs CMFs that have not been calibrated to the specific jurisdiction and may not be representative of the situation under consideration.

Application of Approach-Level Safety Models

There are two sets of possible applications for the approach-level models: (1) they can be used to evaluate the safety performance of an existing roundabout at the approach level, and (2) they can be considered for use in *Highway Safety Manual*-type applications to estimate the expected safety performance at the approach level. Details of these applications are provided in the following sections.

Evaluation of Safety Performance at the Approach Level

Although the approach-level models have been developed to assist with design decisions, the models presented in Chapter 3 also can be used in an EB procedure to estimate the expected safety performance at an approach or number of approaches to an existing roundabout, *provided that the models can be assumed as* representative *of the pertinent jurisdiction or can be recalibrated using representative data from that jurisdiction.* This estimate would be used in screening to compare the performance of the subject roundabout approach to that of other similar approaches. For approaches performing below par from a safety perspective, diagnostic procedures can then be used to isolate any problems and to develop corrective measures.

The EB procedure applied at the approach level would be identical to the example intersection-level procedure presented previously. The models to be used would be those indicated by the shaded rows of Tables 21 through 23.

Consideration of Approach-Level Model Results for HSM-Type Application

The prototype chapter of the HSM documents a crash prediction algorithm that enables the number of total intersection-related crashes per year to be estimated as follows:

$$N_{\text{int}} = N_b\,(CMF_1\, CMF_2 \cdots CMF_n)$$

where

N_{int} = predicted number of total intersection-related crashes per year after application of crash modification factors

N_b = predicted number of total intersection-related crashes per year for base conditions

$CMF_{1\ldots n}$ = crash modification factors for various intersection features, 1 through n

For the prototype chapter, which pertains to two-lane rural roads, a panel of experts selected the base model and CMFs after a review of relevant research findings, including recently calibrated prediction models, the estimated coefficients of geometric variables in these models, and the results of before-after studies.

To apply a similar methodology at the approach level of roundabouts, the first models listed in Tables 21 through 23 (with AADT as the only variable) can be considered in developing base models. And, as noted in Chapter 3, the estimated coefficients for geometric features in the recommended and other approach-level models can be considered in developing CMFs. The CMFs directly related to geometry are shown in Table 24.

Using the previous equation, the effect of a design change can be identified by applying the appropriate CMF. However, caution is advised because many of the variables are correlated, resulting in model-implied effects that may not reflect reality. The correlations should therefore be considered in making final decisions on the CMFs that are to be used in the HSM. To this end, a correlation matrix is provided as Table 25.

Incorporation of Safety Models into Other Documents

The models above have the potential for being incorporated into major documents that guide the transportation profession, including FHWA's *Manual on Uniform Traffic Control Devices* (MUTCD) and the *Highway Safety Manual* under development.

Potential for Use in MUTCD Intersection Control Evaluations

The decision on the form of traffic control for an intersection is based in part on the satisfaction of various warrants provided in the 2003 MUTCD (*43*). As noted in Section 4B.04 of the MUTCD, alternatives to signalized intersection control—including roundabouts (Option K in Section 4B.04)—should be considered, even when one or more signal warrants are met. These alternatives may yield improved safety performance over that of a signalized intersection. The proposed procedures presented in this report for estimating the likely change in safety and operational performance following the installation of a roundabout at an existing conventional intersection (as previously outlined) are anticipated to support this type of evaluation. Given the substantial safety and operational benefits that roundabouts appear to provide in a variety of situations, the language in this section of the MUTCD should be strengthened to further emphasize the need to consider roundabouts as one of these less-restrictive treatments.

Potential for Assimilation into the HSM

As noted previously, there are two potential HSM applications. First, the intersection-level models can be used to estimate the expected crash frequency for network screening applications that are anticipated for Part IV of the HSM, as long as they are representative, or can be recalibrated to be so, of a jurisdiction's roundabouts. Second, the approach-level models and implied CMFs can be *considered* for use in the prototype HSM chapter-type methodology for estimating the expected crash frequency of a roundabout approach (e.g., to estimate the safety implications of a decision to install or improve a roundabout).

The first application would be relatively straightforward to implement, depending on how Part IV is written. The implementation process for the second application is somewhat more complex. The research team recommends that a process similar to that followed in developing the prototype chapter of the HSM be undertaken: the HSM developers—through the various chapter contractors, and perhaps expert panels—would consider the models and CMFs suggested in Chapter 3, *along with all other relevant information*, in finalizing base models and CMFs for roundabouts for application in the pertinent chapters for two-lane and multilane highways.

Application of Operational Models

The operational models and results presented in Chapter 4 form the basis for a proposed revised operational procedure for inclusion in the *Highway Capacity Manual*. A draft procedure has been prepared and attached as Appendix M. This draft is expected to undergo further refinement beyond this project through the activity of the TRB Committee on Highway Capacity and Quality of Service before publication in the next update to the HCM.

The highlights of the proposed HCM procedure include the following improvements over the procedure in the HCM 2000:

- Single-lane model based on an expanded field database
- Guidance on the capacity of double-lane roundabouts, including an approach that is sensitive to lane use
- Procedure for estimating control delay and queues
- Guidance for estimating LOS
- Explanatory text supporting the recommended models
- Sample problems illustrating the use of the procedure

Capacity

Two capacity models are recommended: a model for estimating the capacity of a single-lane entry into a single-lane circulatory roadway and a model for estimating the capacity of the critical lane of a two-lane entry into a two-lane circulatory roadway. Simple, empirical models were chosen for both for two reasons. First, the simple model fit the data as well as or better than any of the more complicated models used internationally. Second, a detailed analysis of geometric parameters at both the microscopic (critical headway and follow-up headway) and macroscopic (goodness-of-fit) level did not reveal a strong relationship that would significantly improve the capacity estimate, other than number of lanes. As a result, the data do not support the use of a more complicated model.

The following exponential regression form is recommended for the entry capacity at single-lane roundabouts:

$$c = 1130 \cdot \exp(-0.0010 \cdot v_c) \qquad (6\text{-}1)$$

where

c = entry capacity (pcu/h)
v_c = conflicting flow (pcu/h)

The exponential model parameters can be calibrated using locally measured parameters as follows:

$$c = A \cdot \exp(-B \cdot v_c) \qquad (6\text{-}2)$$

where

c = entry capacity (pcu/h)
A = $3600/t_f$
B = $(t_c - t_f/2)/3600$
v_c = conflicting flow (pcu/h)
t_f = follow-up headway (s)
t_c = critical headway (s)

The recommended capacity model for the critical lane of a multilane entry into a two-lane circulatory roadway is as follows:

$$c_{crit} = 1130 \cdot \exp(-0.0007 \cdot v_c) \qquad (6\text{-}3)$$

where

c_{crit} = entry capacity of critical lane (pcu/h)
v_c = conflicting flow (pcu/h)

Several influences in the multilane data should be noted:

- The critical-lane data mostly comprise observations in the right entry lane. Because of limited critical left-lane observations, the data were inconclusive in supporting a difference in the capacity between the right lane and left lane.
- The critical-lane data mostly comprise entering vehicles against two conflicting lanes. One site in the data has a two-lane entry and one conflicting lane but has too few observations to draw any conclusions.

The recommended intercept of the critical-lane regression was modified to correspond to field-measured follow-up headways in the critical lane. These headways essentially match the follow-up headways for the single-lane approaches and thus result in the same intercept. The slope of the curve represents a least-squares fit to the data given the intercept constraint.

Control Delay and Queue Models

The recommended control delay model is as follows:

$$d = \frac{3600}{c} + 900T\left[\frac{v}{c} - 1 + \sqrt{\left(\frac{v}{c}-1\right)^2 + \frac{\left(\frac{3600}{c}\right)\frac{v}{c}}{450T}}\,\right] \qquad (6\text{-}4)$$

where

d = average control delay (s/veh)
c = capacity of subject lane (veh/h)
T = time period (h: T=1 for 1-h analysis, T=0.25 for 15-min analysis)
v = flow in subject lane (veh/h)

This model is recommended as a reasonable method for estimating delays at U.S. roundabouts and is consistent with the methods for other unsignalized intersections. Given the low number of U.S. roundabouts currently operating with high delays, there is little ability at the present time to assess the accuracy of this model to predict higher magnitudes of delays. This model should be revisited in the future, where a more reliable estimation technique might become necessary. Because the delay model is the same as is currently used for unsignalized intersections and because the queuing and delay models are related, the current queuing model for unsignalized intersections is recommended for use on roundabout approaches. As discussed in Chapter 4, it may be appropriate to include a "+ 5" factor with some modification for volume-to-capacity ratio. This inclusion is to account for the fact that, at higher volume-to-capacity ratios, vehicles may need to come to a stop and thus incur additional deceleration and acceleration; at low volume-to-capacity ratios, vehicles are more likely to enter without having to come to a complete stop.

Level of Service

The recommended LOS criteria have been given previously in Chapter 4. The recommended thresholds are the same as for other unsignalized intersections because of the similarity in the task required of the driver (finding a gap) and thus in expectations. The LOS for a roundabout is determined by the computed or measured control delay for each lane. The LOS is not defined for the intersection as a whole.

CHAPTER 7

Conclusions and Suggested Research

Based on the findings of this study, roundabouts appear to be successful in a wide variety of environments in the United States. The following sections summarize the major conclusions from this study. In addition, further research is recommended in a number of areas.

Safety Performance

With the exception of conversions from all-way-stop–controlled intersections, where crash experience remains statistically unchanged, roundabouts have improved both overall crash rates and, particularly, injury crash rates in a wide range of settings (urban, suburban, and rural) and previous forms of traffic control (two-way stop and signal). Both types of safety prediction models developed for this project—intersection level and approach level—employ simple model forms that are supported by the available data. These models are of a form that is intended to be suitable for eventual inclusion in the forthcoming *Highway Safety Manual*.

Overall, single-lane roundabouts have better safety performance than multilane roundabouts. The safety performance of multilane roundabouts appears to be especially sensitive to design details.

Operational Performance

Currently, drivers at roundabouts in the United States appear to be somewhat tentative, using roundabouts less efficiently than models suggest is the case in other countries around the world. In addition, geometry in the aggregate sense—number of lanes—has a clear effect on the capacity of a roundabout entry; however, the fine details of geometric design—lane width, for example—appear to be secondary and less significant than variations in driver behavior at a given site and between sites. This finding was tested at two levels: at the microscopic level, in terms of the effect of geometry on critical headways and follow-up headways, and at the macroscopic level, in terms of the overall ability of the model (analytical or empirical regression) to predict capacity.

The resulting recommended operational models are therefore relatively simple and capture only those effects that the data could support. These models are incorporated into an initial draft procedure for the *Highway Capacity Manual*, which the TRB Committee on Highway Capacity and Quality of Service will continue to revise until its eventual adoption.

The proposed models result in lower capacity predictions than have been typically used to date in the United States. Because driver behavior appears to be the largest variable affecting roundabout performance, calibration of the models to account for local driver behavior and changes in driver experience over time is highly recommended to produce accurate capacity estimates. In addition, because the LOS thresholds for other unsignalized intersections appear to be appropriate for roundabouts, similar design standards for all unsignalized intersections (stop-controlled and roundabout) may also be appropriate (e.g., LOS D or E and/or volume-to-capacity ratios of 0.90 to 1.00, depending on the jurisdiction). Standards that allow for higher volume-to-capacity ratios and/or higher delays at roundabouts should be accompanied by local calibration where possible to improve confidence in the capacity and delay estimates. Ideally, such standards should also reflect the need to balance the accommodation of peak hour and/or peak-15-min traffic flows with the safety and accessibility of the intersection for all users.

Geometric Design

Although this project was unable to establish a strong statistical relationship between speed and safety, the importance of controlling speed in roundabout design is well established internationally. Using the current AASHTO-based speed prediction tools as a base, the application of acceleration and deceleration effects appears to significantly improve the ability to predict 85th-percentile speeds entering and exiting a roundabout. This will aid in estimating speeds in the crosswalk area of a roundabout, for example.

The combination of the extensive field observations of critical gap and the revised speed predictions may be used to refine the current intersection sight distance procedure presented in the FHWA Roundabout Guide (*1*). These findings should be considered interim until a more comprehensive study of sight distance needs at roundabouts can be completed.

Anecdotal evidence suggests the importance of considering design details in multilane roundabout design, including vehicle path alignment, lane widths, and positive guidance to drivers through the use of lane markings.

Pedestrian and Bicyclist Observations

The overwhelming majority of the roundabouts observed in this observational study showed very few problems for crossing pedestrians and traversing bicyclists. From a safety perspective, where safety is measured in terms of crashes or in terms of a surrogate such as conflicts, the roundabouts observed performed very well. Out of the 769 pedestrian crossing events and 690 bicyclist events observed from video recordings, there were no observed crashes and only eight observed conflicts (0.5%). The low observation numbers confirm what was found in the crash reports that were collected for this project. Crash reports collected from 139 legs at 39 roundabouts revealed a total of five reported pedestrian crashes and eight reported bicyclist crashes across all sites over a mean reported crash history period of 3.8 years per site.

Another approach to measuring the risk to pedestrians and bicyclists is to observe the interactions between pedestrians/bicyclists and motorists. The majority of the analyses in this study were focused around these interactions. The major findings from these analyses can be summarized as follows:

- Exit lanes appear to place crossing pedestrians at a greater risk than entry lanes. Motorists were less likely to yield to pedestrians on the exit side (38% of the time) compared to the entry side (23% of the time). Pedestrians and bicyclists were also more likely to hesitate when starting to cross from the exit side versus the entry side. Approximately 22% of the pedestrians that started crossing from the entry side hesitated, compared to 31% of those that started from the exit side. Similarly, 21% of the bicyclists hesitated when starting from the entry side compared to 29% when starting from the exit side.
- Two-lane legs are more difficult for pedestrians to cross than one-lane legs, primarily because of the non-yielding behaviors of motorists. On one-lane legs, 17% of the motorists did not yield to a crossing/waiting pedestrian. On two-lane legs, the non-yielding percentage was 43%. The lack of yielding was perhaps reflected in the observed pedestrian behaviors. Single-lane legs resulted in hesitation crossings 24% of the time, while two-lane legs produced hesitations 33% of the time.
- Roundabouts result in the type of behaviors expected when compared to other types of intersections and levels of traffic control. Roundabouts, which are under yield control, produced motorist and pedestrian behaviors that were between the behaviors observed at crossings with no traffic control and those observed at crossings with signal or stop control. Motorists not yielding to pedestrians ranged from 48% at uncontrolled locations to 32% at roundabouts to 15% at signalized locations to 4% at stop-controlled sites. Crossings in which the behavior of the pedestrian was considered to be normal were 70%, 85%, 90%, and 100% for uncontrolled, yield-controlled, signalized, and stop-controlled locations, respectively. However, there was no practical difference in the crossing pace of pedestrians between the various types of traffic control.
- Bicyclists appear to have very few problems interacting with motorists and maneuvering through a roundabout. Bicyclists approaching a roundabout were most often positioned near the edge of the travel lane (73%), while bicyclists in the circulating lane most often possessed the lane (83%). The problems that were identified were the result of inappropriate behaviors on the part of the bicyclists. One of the conflicts occurred because a circulating bicyclist was riding on the outside of the travel lane as opposed to taking the lane, which resulted in the bicyclist almost being clipped by an exiting vehicle. The other conflict involved a wrong-way rider who was entering the roundabout on the exit lane.

In summary, the findings of this research did not find any substantial safety problems for non-motorists at roundabouts, as indicated by there being few reported crashes and a very small number of observed conflicts. At the same time, the findings have highlighted some aspects of roundabouts where pedestrian and bicyclist ability to use the roundabout may be compromised as use of the roundabout by all modes and their subsequent interactions are greater than studied herein or where such interactions increase over time (i.e., as vehicle traffic and/or pedestrian traffic increases). For example, care must be taken to ensure that vehicles yield to waiting or crossing pedestrians. An emphasis needs to be placed on designing exit lanes to improve upon the behaviors of both motorists and pedestrians. And multilane roundabouts may require additional measures to improve upon the behaviors of motorists, pedestrians, and bicyclists.

The specific countermeasures required to change the observed motorists' behaviors may include changes in design, changes in operations, and/or targeted enforcement and education. Design changes could include reduction in the exit radius, reduction in lane widths, and/or relocation of the crosswalk. Operational changes could include static warning signs, real-time warning devices that are activated when a pedestrian is present, and/or some form of pedestrian-actuated signalization. Enforcement and education could focus on improving user compliance with existing rules of the road. Any implemented countermeasure should be evaluated to determine if it does indeed result in behavioral changes that increase safety and mobility for nonmotorists. The results of such evaluations should be used to change the design guidance that is currently available.

No exposure data were available for the condition before a roundabout was installed at each study location, so it is unknown whether pedestrians have altered their travel patterns because of the presence of a roundabout.

This study did not address the accessibility of roundabouts for pedestrians with visual impairments. As noted in the introduction, a separate research project (NCHRP 3-78) was developed to specifically address this issue. In addition, the U.S. Access Board is continuing to be active in proposing guidelines for accessible rights-of-way; the reader is encouraged to stay abreast of this developing area.

Suggested Research

Even though the scope of this study was broad, the research team uncovered a variety of topic areas that warrant further research beyond what could be completed in this study. In some cases, the research may simply involve further analysis of the data collected as part of this study. In other cases, additional data collected at a later date may be needed to provide adequate sample size and diversity.

Updated Operational Models

The operational models developed under this study are believed to be the best analysis of the data available in the United States at the time of this study. However, the research was conducted in the full knowledge that few sites in the United States have reached capacity, and most of those are at capacity for only short periods of time. In addition, the diversity of multilane roundabouts operating at capacity is particularly thin. Therefore, further study is needed at a future date to (1) collect data at more sites operating at capacity for longer durations, (2) determine whether capacities have changed over time, and (3) expand the diversity of sites included in such a study, particularly among multilane roundabouts.

A draft research problem statement on this topic is included in Appendix N.

Updated Safety Models

Unsurprisingly, the sample of available data was found to be such that better safety models could not be calibrated. While what was accomplished in this regard is still quite useful, it would nevertheless be worthwhile to build on the database and modeling effort for this project. For example, for the approach-level models, several variables appeared to influence roundabout safety, but a larger sample would be required to resolve the correlations that were quite evident among several of these variables. And it would be very desirable to have a large enough sample to develop approach-level models for *injury* crashes.

The larger sample could be built through a combination of assembling complete data for more roundabouts and assembling data for additional years for those roundabouts currently in the database. In assembling data for newly added roundabouts, attention should be paid to ensuring that there is variation in the variables of interest. At the same time, the sample for evaluating safety before and after roundabout installation could also be expanded because, undoubtedly, most of the intersections added to the modeling database will have been converted from some other form of control. A before-after evaluation based on a larger sample would improve knowledge of the geometric and operational conditions that better favor roundabout construction and safety.

Given the rate at which new roundabouts are being constructed, the research team recommends that the safety analysis, both the modeling and before-after evaluation, be revisited 5 years after the data for this project were collected, or approximately 1 year from the date of this report. To this end, care should be taken to preserve the safety data and analysis worksheets developed for this project, so that the task of building on them would be considerably easier than starting afresh.

Intersection Sight Distance

This project revised the gap acceptance parameters used in the modeling framework for intersection sight distance originally established in the FHWA Roundabout Guide. However, this project did not have the resources to more fully study the entire sight distance methodology, including the fundamental question of how much sight distance is appropriate and how much sight distance may be excessive. This question is relevant, as some international studies have suggested that excessive sight distance at roundabouts leads to higher crash frequencies (*4*). Therefore, a new research effort is recommended, similar in scope to that conducted for intersection sight distance at conventional intersections, to address this fundamental parameter.

Additional Topics

A variety of additional topics that could not be covered within the scope and budget of this project are also worth exploring further:

- Relationship between vehicle speeds and roundabout safety
- Relationship between design details and safety at multilane roundabouts
- Relationship between safety and illumination
- The operational effects between nearby traffic control devices and roundabout operations

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Appendixes

The following appendixes have been published as *NCHRP Web-Only Document 94,* available on the TRB website [http://www.trb.org/news/blurb_detail.asp?id=7274]:

- Appendix A, Literature Review on Safety Performance
- Appendix B, Literature Review of Operational Models
- Appendix C, Site Inventory
- Appendix D, Pedestrian and Bicycle Analysis Details
- Appendix E, Summary of Goodness of Fit Measures and Statistical Terms
- Appendix F, Statistical Testing of Intersection-Level Safety Models
- Appendix G, Definitions for Estimating Fastest Vehicle Paths
- Appendix H, Statistical Testing of Approach-Level Safety Models
- Appendix I, Statistical Testing of Speed-Based Safety Models
- Appendix J, Operations Appendix
- Appendix K, Pedestrian Analysis Tables
- Appendix L, Pedestrian and Bicycle Images
- Appendix M, Draft Highway Capacity Manual Chapter 17
- Appendix N, Research Problem Statement

Abbreviations and acronyms used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation