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Nevada Roundabout Implementation Guidelines

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Disclaimer

Nevada Roundabout Implementation Guidelines

Final Report

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Center for Advanced Transportation Education and Research (CATER),
University of Nevada, Reno

Authors

Samuel Kofi Ahiamadi
Zong Tian, Ph.D., P.E. (PI)
Adrain Reed Gibby, Ph.D., P.E.

Center for Advanced Transportation Education and Research
Department of Civil & Environmental Engineering
University of Nevada, Reno
Reno, NV 89557

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List of Abbreviations

FHWA – Federal Highway Administration
NCHRP – National Cooperative Highway Research Program
HCM – Highway Capacity Manual
ROW – Right of Way
LOS – Level of Service
AWSC – All-Way Stop Control
TWSC – Two-Way Stop Control
ADT – Average Daily Traffic
DOT – Department of Transportation
AADT – Annual Average Daily Traffic
MUTCD – Manual on Uniform Traffic Control Devices
NDOT – Nevada Department of Transportation
MDOT – Maryland Department of Transportation
WisDOT – Wisconsin Department of Transportation
PennDOT – Pennsylvania Department of Transportation
ODOT – Oregon Department of Transportation
FDOT – Florida Department of Transportation
ADOT – Arizona Department of Transportation
WSDOT – Washington State Department of Transportation
KDOT – Kansas Department of Transportation
NYSDOT – New York State Department of Transportation
MoDOT – Missouri Department of Transportation
MnDOT – Minnesota Department of Transportation

Executive Summary

In 1990 Nevada constructed the first modern roundabout in the U.S. and since then several roundabouts have been constructed with most operating successfully; the U.S. currently has over forty-five states with at least one modern roundabouts. Due to their safety performance and operational efficiency roundabouts have gained popularity among transportation practitioners and the general public. As an intersection control, roundabouts operate efficiently with minimal cost under a wide variety of conditions. Based on United States and international experience, roundabouts are known to reduce delay, total crashes and crash severity for intersections with low to medium traffic volume. After the introduction of roundabouts into the U.S., several national research efforts were initiated and the findings provided important information for analysis and design. The most prominent research publications include: 1) National Cooperative Highway Research Program (NCHRP) Report 672 - Roundabouts: An Information Guide, Second Edition, co-funded by Transportation Research Board (TRB) and Federal Highway Administration (FHWA), 2) NCHRP Report 572 - Roundabouts in the United States funded by the TRB 3) Highway Capacity Manual (HCM) 2010 (chapters 21 and 33) funded by the TRB. After the national initiative, several states developed their state specific guidelines which adopted most of the national findings but sometimes deviated depending on the states' experience and research. Nevada, despite having about 81 roundabouts, lacks a statewide guideline for selection/installation. In anticipation of more roundabouts being constructed in the state, the Nevada Department of Transportation (NDOT) funded this research project, *Development of Guidelines for Implementing Roundabouts in Nevada*.

A comprehensive literature review revealed several important roundabout findings including, 1) safety implications, 2) roundabout analysis tools, 3) site selection guidelines and 4) geometric design considerations. This was completed by gathering information from the NCHRP reports, state agency guidelines, and major research publications. The research team collected data from nine roundabouts in northern and southern Nevada and extracted information leading to the determination of the critical and follow-up headways for Nevada drivers. Using the critical and follow-up headways obtained, the capacity models given in the HCM, 2010 were calibrated. Two roundabout analyses software, SIDRA Solutions and Highway Capacity Software (HCS) were compared to select the better tool for modeling roundabout performances in Nevada. A roundabout selection/installation guideline was also developed for use by traffic engineers based on information obtained from the literature review and modeling tasks.

The key findings and recommendations from this research include the following:

1. There is no specific (comprehensive) guideline or selection process for installation of roundabouts.
2. Nevada drivers have a critical headway that is consistent with critical headways obtained for other states as represented in the NCHRP 3-65 Project.
3. Nevada drivers exhibit a follow-up headway that is lower than the follow-up headways obtained for other states as represented in the NCHRP 3-65 Project.
4. The following are average values obtained for Nevada critical and follow-up headways.
    a. For single-lane roundabouts: critical headway = 3.9 seconds, follow-up headway = 2.9 seconds
    b. For double-lane roundabouts, (left lane): critical headway = 4.9 seconds, follow-up headway = 2.9 seconds
    c. For double-lane roundabouts, (right lane): critical headway = 4.8 seconds, follow-up headway = 2.9 seconds
5. From the HCM 2010 recommendation on calibration of the capacity equations, and using the critical and follow-up headways from above, the capacity (C) of Nevada roundabouts is related to the conflicting flow rate (v_c) by the following relationships:
    a. For single-lane roundabouts: $C_{pce} = 1{,}230e^{(-0.67\times10^{-3})v_{c,pce}}$
    b. For two-lane roundabout, (left lane): $C_{e,L,pce} = 1{,}231e^{(-0.95\times10^{-3})v_{c,pce}}$
    c. For two-lane roundabout, (right lane): $C_{e,R,pce} = 1{,}221e^{(-0.92\times10^{-3})v_{c,pce}}$
6. A flowchart has been developed to assist engineers in making a preliminary decision on whether to use roundabouts at intersections. It is recommended that engineers use this chart as an initial step in the roundabout consideration process before embarking on detailed designs.
7. SIDRA Solutions is recommended for use as the preferred roundabout analysis software for Nevada.

1 Introduction

1.1 Background

Intersections play a significant role in the efficient movement of vehicle, pedestrians, and bicyclists' traffic on roadways. To achieve efficient flow, intersections need to accommodate traffic volumes and flow patterns. Selecting the best type of intersection control is often challenging for transportation professionals, since a poor selection can result in safety issues and/or operational failures prompting user complaints. Unique characteristics of an intersection are considered to determine which control type is suitable. The four common controls for busy intersections in the United States (U.S.) are: 1) the traffic signal, 2) Two-Way stop control (TWSC), 3) All-Way stop control (AWSC) and recently 4) modern roundabout.

Modern roundabouts were introduced into the U.S. in 1990 and since then, the numbers have increased substantially with over 45 states having installed at least one roundabout. The first one was constructed in Summerlin, Las Vegas- Nevada (*1*). Previously, traffic circles existed in the U.S. but lost favor when increased traffic volumes resulted in operational failures and increases in crashes. Most traffic circles were therefore replaced with traffic signals or stop controls. The few traffic circles that remained in the U.S. were predominately in residential areas mainly to discourage through trips. The United Kingdom (U.K.) however found a solution to operational failures experienced with the old traffic circles by modifying the operational design which lead to improved safety (*2*). The main design changes were allocating the right-of-way (priority) to the circulating traffic and introducing "yield" control on all entry approaches. Modern roundabouts (also referred to as roundabouts) then evolved and over the years spread to other countries.

Roundabouts are generally known as efficient intersection control type that improves safety by eliminating head-on and angle crashes, reduces vehicular delays and stops, and lowers emissions compared with other controls at low to medium traffic volumes. Modern roundabouts also have the advantage of reducing higher speed crashes, hence reducing crash severity. Earlier, there was little domestic knowledge and data on roundabout operations and design in the U.S., therefore, transportation professionals reliance heavily on foreign knowledge. Initial guideline development efforts were made by three states Florida, Maryland and Oregon who borrowing heavily from Australian and European guidelines. The FHWA initiated the first nationwide effort in the U.S. that produced a roundabout applications guideline titled “Roundabouts: An Information Guide” (*3*) in 2000. After the FHWA Roundabout Guide was published, more than fifteen states have developed their statewide guidelines as of 2011. Generally, the state-specific guidelines follow the FHWA guide and sometimes with modifications to address state uniqueness. After several years of research, a second edition of the; “Roundabouts: An Information Guide” was published in 2010 as NCHRP 672 through joint sponsorship from FHWA and American Association of State Highways and Transportation Officials (AASHTO). This guide reflects the most current information on roundabout experience in the U.S.

Nevada, though pioneered the construction of modern roundabouts in the U.S., has no formal statewide guideline for roundabout installation. An earlier study in southern Nevada by the University of Nevada, Las Vegas (*4*,*5*), confirmed that roundabouts improved safety at intersections and anticipated that the number of roundabouts would continue to increase in Nevada. The absence of a statewide guideline is, therefore, likely to result in inconsistencies in the design and installation since agencies would likely rely on the FHWA guide and professional judgment with the possibility of major variations.

1.2 Objective and Scope

The main purpose of this research was to develop roundabout installation guidelines unique to the State of Nevada. Specific objectives of this research are: 1.) Measure the critical headway and follow-up headway for Nevada drivers. 2.) Conduct roundabout operational analyses using available tools such as HCS and SIDRA and 3.) Develop site selection guidelines to assist transportation engineers in their consideration of potential roundabout locations (before detailed analysis and design phase). This report discusses issues with roundabouts based on national and international studies with subjects of particular interest being: a) critical headway and follow-up headway, b) guidelines relating to installation criteria, c) safety issues for vehicles, pedestrians and bicyclists, and d) roundabout operational analysis software. Chapter 2 is a comprehensive literature review of modern roundabouts; Chapter 3 explains the operational data acquisition for Nevada roundabouts. Chapter 4 described the guidelines developed for suggesting circumstances when roundabout should be used. Finally, Chapter 5 contains the conclusions, recommendations and implementation of these guidelines.

2 Literature Review

2.1 Introduction

Modern roundabouts evolved from existing old traffic circles that experienced operational failures and high crash rates as traffic volumes increased. The “priority rule” assigning right-of-way (priority) to circulating traffic on roundabouts was first introduced in the United Kingdom in 1966 (*6*) and resolved the problems experienced with old traffic circles. The priority rule together with the “Yield” sign for the vehicles entering the roundabout led to improvements in the operations experienced with modern roundabouts. With improved operational and safety features, modern roundabouts spread to France, Germany, Australia, etc (*2*) and now the U.S. Over the years and following several research findings, roundabouts are generally considered to offer several advantages over signalized and stop-controlled alternatives. Advantages of roundabouts include lower maintenance and operating costs, better overall safety performance (40 percent reduction for all crashes and 80 percent reduction for injury crashes) (*7*), reduction in delay (from 7.2 seconds to 1.3 seconds per vehicle) (*8*) and reduction in service time (from 18.1 s to 0.53 seconds per vehicle) (8). Roundabouts thus result in shorter queues compared to other controls especially during off-peak hours. They provide better speed management, reduce air and noise pollution and create opportunities for community enhancement features like landscaping. Roundabouts can also operate more efficient and safer under a wide variety of conditions including; 1) reduced approach speeds on all approaches (≤30 mph), 2) significant variations in peak and off-peak traffic volumes and 3) skewed approaches can be better accommodated than other control types. (*2*).

This chapter discusses eight topics that thoroughly explore the modern roundabout based on information extracted from current policies, practices, guidelines and standards published in the U.S. and elsewhere. These topics are grouped into: 1) key features of modern roundabouts, 2) modern roundabout guide development in the U.S., 3) safety performance of roundabouts, 4) performance measures for roundabouts, 5) critical headways and follow-up headways, 6) site selection guidelines, 7) roundabout installation considerations and finally 8) geometric design considerations.

2.2 Key Features of Modern Roundabouts

Figure 1 shows the key features of a modern roundabout *(3)*. In this figure, appropriate geometric features that promote slower and more consistent speeds for all movements are shown.

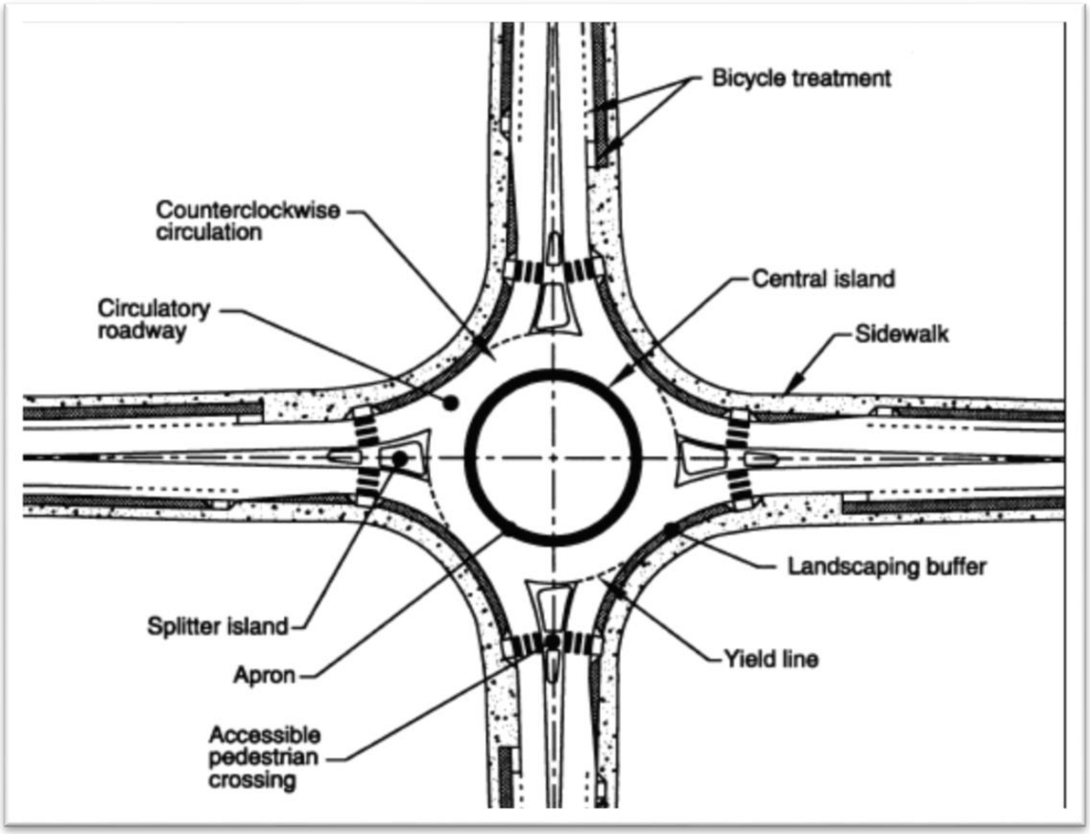


Figure 1: Drawing of Key Roundabout Features

Source: NCHRP Report 672

The following is a brief description of the key components of the modern roundabout *(*3) shown in Figure 1.

Central Island: The raised area in the center of a roundabout around which traffic circulates, typically circular in shape.

Splitter Island: A raised or painted area on an approach used to separate entering and exiting traffic, deflects and slows entering traffic, and provides storage space for pedestrians crossing the road in two stages.

Circulatory Roadway: The circular path used by vehicles to travel in a counterclockwise fashion around the central island

Apron: The mountable portion of the central island adjacent to the circulatory roadway required to accommodate the wheel tracking of large vehicles. Sometimes the apron is provided on the outside of the circulatory roadway.

Entrance Yield Line: A pavement marking used to mark the point of entry from an approach into the circulatory roadway and is generally marked along the inscribed circle. Entering vehicles must yield to any circulating traffic coming from the left before crossing this line into the circulatory roadway.

Accessible Pedestrian Crossings: Should be provided at all roundabouts. The crossing location is set back from the yield line, and the splitter island is cut to allow pedestrians, wheelchairs, strollers, and bicycles to pass through.

Bicyclist Treatments: Provides bicyclists the option of traveling through the roundabout either as a vehicle or as a pedestrian, depending on the bicyclist's level of comfort.

Landscaping Buffer: Provided at most roundabouts to separate vehicular and pedestrian traffic and assist with guiding pedestrians to designated crossing locations. Landscaping buffers aid the visually impaired in crossing and also significantly improves the aesthetics of the roundabout.

2.2.1 Distinguishing Roundabouts from Other Circular Intersections

Circular intersections include "old-style" rotaries, neighborhood traffic circles, and modern roundabout but they are frequently confused *(*3*)* even though there are significant differences.

Table 1 compares the characteristics of the three types of circular intersections *(*9*)*. Roundabouts typically slow all vehicles to speeds that are between 10 and 30 mph. The roundabouts' geometry and use of channelized approaches (splitter islands and an outside curb) help deflect vehicles as they approach and enter the circulating roadway. Another key feature of modern roundabouts is "Yield" signs on all entries. Drivers approaching the circular intersection must yield at the entry if an acceptable gap is not available to enter the circulating roadway. If an acceptable gap is available, the driver may proceed into the circulatory roadway without stopping. However, drivers stopping at the yield line when acceptable gaps exist can have a negative effect on capacity.

Table 1 Comparison of Old-Style Rotaries, Neighborhood Traffic Circles and Modern Roundabouts

	Modern Roundabout	**Neighborhood Traffic Circles**	**Old-style Rotary**
Traffic Control	Yield on all entries	Stop control, yield control, or no control	Stop control, no control, sometimes signalized
Priority	Circulating vehicles have the right of way	Circulating vehicles have the right of way	Some have circulating vehicles yielding to entering vehicles
Deflection	Entry angles create deflection to control speeds	Entry angles close to 90o	Entry angles close to 90o
Speed	Low speeds (< 25 mph normally)	Low speeds (< 25 mph normally)	Higher speeds (> 25 mph)
Diameter	Small inscribed circle diameters (80 ft - 200 ft) Mini roundabout (45 ft – 80 ft)	Center island diameters (< 20 ft)	Large inscribed circle diameters (> 300 ft)
Pedestrians	Access allowed only across the approach legs	Access allowed only across the approach legs	Access can be allowed to the center island and across the approach legs
Parking	No parking within the circulating roadway	No parking within the circulating roadway	Parking is sometimes allowed within the circulating roadway
Circulation	All vehicles travel counterclockwise and pass to the right of the center island	Some turning traffic may be allowed to pass to the left of the center island	Some traffic may be allowed to pass to the left of the center island

Source: Planning level Guidelines for Modern Roundabouts: Iowa State University

2.2.2 Categories of Roundabouts

Roundabouts are categorized according to size and environment to differentiate their design and operational characteristics within different contexts. The 2010 FHWA Guide (9) categorizes roundabouts into three basic groups, namely: 1) mini-roundabouts, 2) single-lane roundabouts and 3) multilane roundabouts. The 2000 FHWA Guide (3) used site environment as part of the roundabouts description and hence obtained six categories: 1) Rural single-lane roundabouts 2) Rural double-lane roundabouts 3) Urban single-lane roundabouts 4) Urban double-lane roundabouts 5) Urban compact roundabouts and 6) Mini-roundabouts.

Table 2 is a summary of some basic design and operational elements for each of the three broad roundabout categories described in the new FHWA guide (9). Other categorizations of roundabouts based on special features are defined in the British Highways Agency Manual (10). Examples include: grade separated roundabouts (usually at grade separated interchanges), signalized roundabouts (one or more approaches have a signal control installed), and double roundabouts. Figure 2 is an illustration of double roundabout.

Table 2 Roundabout Category Comparison

Design Element	Mini-Roundabout	Single-Lane Roundabout	Multi-Lane Roundabout
Desirable maximum entry design speed	15 to 20 mph (25 to 30 km/h)	20 to 25 mph (30 to 40 km/h)	25 to 30 mph (40 to 50 km/h)
Maximum number of entering lanes per approach	1	1	2+
Typical inscribed circle diameter	45 to 90 ft (13 to 27 m)	90 to 180 ft (27 to 55 m)	150 to 300 ft (46 to 91 m)
Central island treatment	Fully traversable	Raised (may have traversable apron)	Raised (may have traversable apron)
Typical daily service volume on 4-leg roundabout below which may be expected to operate without requiring a detailed capacity analysis (veh/day)	Up to approximately 15,000	Up to approximately 25,000	Up to approximately 45,000 for two-lane roundabout

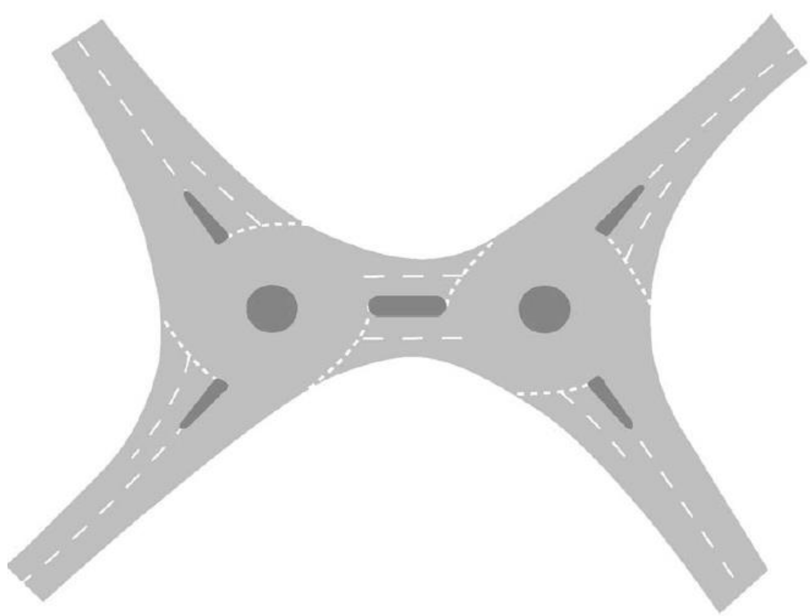


Figure 2: Illustration of a Double Roundabout with Short Central Link

Source - British Manual for Design or Roads and Bridges

2.3 Roundabout Design Guideline Development in the United States

According to the Insurance Institute of Highway Safety (IIHS) (*11*), there are currently over 2000 roundabouts in the U.S. as of December 2010, of which there are about 81 in Nevada. Roundabout construction was opposed in several states in the early 90’s e.g. Kansas, Washington, Arizona, Wisconsin, Maryland, Idaho and New York. The opposition was due mainly to the misconception that roundabouts are unsafe and difficult for large trucks to negotiate. However a research study (*12*) found that public acceptance percentages generally increased from 36 percent to over 70 percent within one year after construction in several states. Currently about 45 states (*13*) have used roundabouts as a control for at least one intersection since experiences in the U.S. confirmed several advantages. When roundabouts were introduced into the U.S., there was reliance on international sources for guidance mostly from the UK, Australia, France and Germany. Before major national efforts in developing guidelines were completed, Maryland (1995) (*14*), Florida (1996) (*15*), and Oregon (1998) (*16*), developed their own roundabout guidelines. National efforts at promoting and gathering data on roundabouts led to several research efforts some of which are listed below:

- The FHWA Guidebook published in 2000 with the publication FHWA-RD-00-0067 titled “Roundabouts: An Informational Guide” (3) served as a one stop reference and foundation for most state guidelines developed after the year 2000.
- NCHRP 3-65 Project “Applying Roundabouts in the United States” examined the safety and operational impacts of roundabouts and produced updated design criteria. It was published as NCHRP Report 572, titled “Roundabouts in the United States” (*17*).
- NCHRP 3-65A produced a second edition of the “Roundabout: An Informational Guide in 2010, published as NCHRP Report 672, (9). It is an update of the original FHWA guidelines with additional information based on U.S. research findings.
- Another recently completed project NCHRP 3-78, “Crossing Solutions at Roundabouts and Channelized Turn Lanes for Pedestrians with Vision Disabilities”, recommended a range of geometric designs, traffic control devices, and other treatments that improves pedestrian crossings at roundabouts and channelized turn lanes usable by pedestrians with impaired vision.

Following the federal research effort, several states including Kansas (*18*), Arizona (*19*), California (*20*) Iowa (*21*), Wisconsin (*22*), Idaho (*23*), New York (*24*), Washington (*25*), Utah (*26*), Missouri (*27*), Minnesota (*28*) and Pennsylvania (*29*) have developed guidelines to reflect their individual state's unique needs along with newer research findings. Some state guidelines developed after 2000 contained some deviations from the original FHWA roundabout guide although most states followed the guide closely.

2.4 Safety Performance of Roundabouts

Roundabouts have been proven as a good strategy for improving intersection safety through the elimination of most conflict types and speed reduction. Understanding the interaction of the various components like geometry, design elements and traffic exposure is important to these safety demands. To compare the safety performance of roundabouts with other intersection traffic controls, typical crash rates for similar types of intersections were obtained and compared with roundabout crash data (*30*). The safety reputation of roundabouts is well documented internationally and is confirmed by U.S. studies that show that converting signal or stop controlled intersections to roundabouts generally reduced crashes significantly (*31*). Persuad et al (*32*,*33*), reported over 50 percent reductions in incapacitating injuries, fatalities and vehicle crashes for roundabouts. This is largely attributed to reduced speeds and elimination of conflicts including: head-on, left turns against opposing vehicles, rear end and right turning collisions with pedestrians and bicyclist in roundabouts. Speed reduction translates into reduction in crash frequency and severity since drivers have sufficient time to react to emergencies.

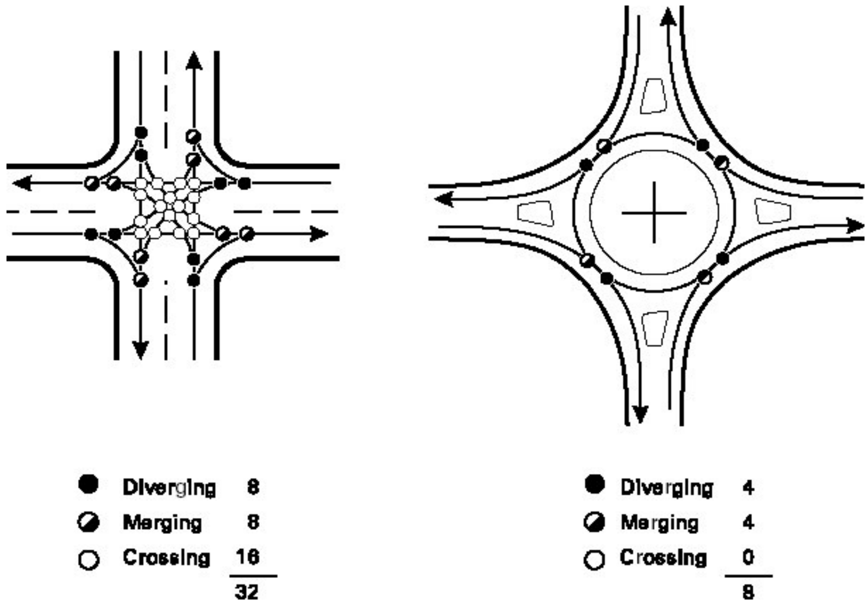


Figure 3: Comparison of Vehicle-Vehicle Conflict Points for Four-Legged Intersection and Roundabout
Source: NCHRP Report 672

Figure 3 (3) is an illustration of the conflict points on a roundabout compared to stop controlled or signalized intersections. In this figure, the vehicle-vehicle conflict points for a standard four-legged intersection are 32 points compared to 8 points for a roundabout. Just as for conventional intersections, safety aspects on double-lane roundabouts are different since they generate additional conflict points due to lane changes on the roundabout and the two approaches lanes.

2.4.1 Vehicle-Vehicle Crashes

Eisenman et al (34) studied the safety and operational performance of roundabouts in the U.S. and observed that out of 35 roundabout sites studies, 28 showed reduction in crashes. This amounts to 80 percent of the sites with a decrease in total crashes resulting in an average crash reduction of 47 percent. Earlier studies carried out by Flannery and Elefteriadou (35) also showed crash reductions consistent with those by Eisenman (34). Their study consisted of eight single lane roundabouts in Florida and Maryland. The analysis of the before and after crash data showed a reduction in both the crash frequency and crash rate. They also found the injury frequency and rate decreased which is consistent with what was reported in other countries. Burley (30) in his study on Australian roads compared average safety performance data for arterial intersections controlled by roundabouts and traffic signals, the safety performances are summarized in Table 3 and Table 4.

Table 3 Summary of Casualty Crash Rates

Arterial Road/ Arterial Road Intersections	Mean Casualty Crash Frequency[1] (Crash/Intersection/year)	
	Traffic Signals	Roundabout
Inner Melbourne[2]	2.11	1.08[4]
Outer Melbourne[3]	2.00	1.01
Country Victoria (Rural Cities and Town)	0.95	0.47

Notes:
1. *Based on accident data for the period 2000 – 2002 for urban locations and 1998 to 2002 for non urban locations.*
2. *Municipalities of Melbourne, Yarra, Port Phillip, Stonington, Boroondara, Glen Eira, Bayside, Hobson's Bay, Maribyrnong, Mooney Valley, Brimbank, Moreland, Darebin, Banyule, Whitehorse, Monash and Kingston.*
3. *Municipalities of Cardinia, Casey, Frankston, Dandenong, Hume, Knox, Manningham, Maroondah, Melton, Mornington Peninsula, Nillumbik, Whittlesea, Wyndham and Yarra Ranges.*
4. *Low sample size – 37 sites.*

Table 4 Summary of Casualty Crash Exposure Rates

Arterial Road Intersections – Melbourne Urban Area	Mean Casualty Crash Rate[1] (Crash/Intersection/year/10^7 entering vehicles)	
	Traffic Signals	Roundabout
Inner Melbourne	1.41	1.12
Outer Melbourne	1.73	1.40
Country Victoria (Rural Cities and Town)	1.68	1.26

Note: Based on accident data for the period 1992 to 1994. Data shown is the average values for divided and undivided roads without trams.

Results in Table 3 shows that the average casualty crash rate at roundabouts is about 50 percent lower than the rate for signalized intersections for all road environments studied. Table 4 also shows about 20 – 25 percent reduction in crash exposure rate for roundabouts compared to traffic signals for all road environments studied.

2.4.2 Pedestrian – Vehicle Crashes

The safety of pedestrians at unsignalized intersections is of primary concern. Results of studies in the U.S. and other countries indicate that the conversion of other intersection control types to roundabouts usually reduce pedestrian-vehicle conflicts. For roundabouts, pedestrian crossings are restricted to designated locations on the approaches and exits. Pedestrian safety is more an issue of perception than real risk. Even though pedestrian safety at roundabouts is based mainly on international experience and U.S. studies suggest many pedestrians do not perceive roundabouts as safe. Yet, compared to intersections with other controls, roundabouts will likely improve pedestrian safety, especially for crossing the major street since approach speeds are lower without unexpected right or left-turning movements. (18, 30). Harkey and Carter (*36*) studied the characteristics of pedestrian and bicyclist interactions with motor vehicles at roundabouts and made some observations. They studied 769 pedestrian crossing events and the key observations are noted below:

- 0.5 percent conflict between pedestrians and motor vehicles
- The exit leg appears to place crossing pedestrians at a greater risk than entry legs. This is because motorists were less likely to yield to pedestrians on the exit leg than on the entry leg. Both pedestrians and bicyclists were also more likely to hesitate when starting to cross from the exit leg compared to the entry leg.
- Single-lane approaches were more favorable for crossing pedestrians than double-lane approaches due to many motorists not yielding to pedestrians. Single-lane approaches showed 17 percent non-yielding drivers compared to 43 percent on two-lane approaches.
- A higher percentage of drivers did not yield to pedestrians on roundabouts compared to other forms of intersection controls.

Harkey and Carter (36) concluded that overall, there were few problems for pedestrians and bicyclists on the majority of roundabouts they studied. From the FHWA Design Guide (*3*), roundabout splitter islands provide refuge to pedestrians and allow them to cross one direction of traffic at a time. However, the crosswalks are set back from the yield line creating additional walking distance as they usually occur between the first and second vehicles in the queue. Both situations are unusual for U.S. pedestrians. Stone et al. (*37*) observed differences in pedestrian – vehicle right of way rules for roundabouts and other intersection controls.

From the design perspective, roundabouts result in fewer potential vehicle - pedestrian conflict points compared to other intersection types with studies in Australia showing an average of 0.02 pedestrian crashes per roundabouts per year (34). The findings also showed that severity of crashes involving pedestrians were lower than other intersection control types, with 2 percent of crashes resulting in fatalities and 66 percent being lower severity type crashes (34). Fritzpatrick et al (*38*) have developed guidelines for selecting pedestrian crosswalk type at unsignalized intersections and mid-block crossing locations. The procedure uses key input variables such as pedestrian volume, street-crossing width, and vehicular traffic volume.

2.4.3 Pedestrians with Vision Impairment

Pedestrians with visual disabilities are a major concern at roundabouts for the reasons that, some drivers do not yield to pedestrians at roundabouts (34). Blind pedestrians have difficulty when it comes to detecting gaps using the sound of vehicles and also because the acceptable gaps might not be frequent enough for them to cross. Traffic sound at roundabouts can prove to be ambiguous as Inman et al. (*39*) found in their research. They found that circulatory vehicles can mask the sound of entering and existing vehicles making it difficult to identify a safe crossing gap. Ashmead et al. (*40*) suggested roundabouts pose further challenges to blind pedestrians because of their curvilinear layout. Wadhwa (*41*,35) reported that identifying the location and direction of the crosswalk can be a major challenge for the blind since most sidewalks leading to the crossings rarely follow a straight path. Wadhwa suggested a number of solutions to aid blind pedestrians in crossing roundabouts safely. These include:

- Minimize crossing distances by making crosswalks as straight as practical
- Provide clues to help them identify the street ahead and determine safe crossing periods. Examples are traffic sound, textural difference between the street and side walk, detectable underfoot warnings, and audible informat
- Provision of pedestrian activated signals equipped with locator tones
- Design elements e.g., stop lines set back from cross walks, extending medians into crosswalks, etc.

Inman et al. (39) suggested that since motorists tend to stop better upstream, crosswalks should be moved two to three vehicle lengths from the inscribed circle. This ensures vehicles yielding to pedestrians do not obstruct the circular roadway though some exiting drivers might be unwilling to stop. A recently completed national project NCHRP 3-78, provides in-depth guidelines to improve accessibility to roundabouts and similar facilities for the visually impaired. The report is published under the title NCHRP Report 674: “Crossing Solutions at Roundabouts and Channelized Turn Lanes for Pedestrians with Vision Disabilities".

2.4.4 Bicyclist – Vehicle Crashes

The number of conflicts for bicyclists depends on the roundabout design. At roundabouts bicyclists have the option of travelling through as a motor vehicle, or dismount and traverse as a pedestrian. A bicyclist, therefore, faces about the same number of conflicts as a driver or a pedestrian. However, when travelling as a motor vehicle in multi-lane roundabouts bicyclists face additional conflicts due to overlapping paths with motor vehicles, because bicyclists typically ride on the right side of the road. If there are no separate bicycle facilities, bicyclists mix with motor vehicles on the roundabout and experience the same conflicts as vehicles. Sometimes, the number of conflicts could be higher than for bicyclist, due to speed differences and visibility between bicyclists and motor vehicles (11). Harkey and Carter (36) in a study concluded that, 73 percent of bicyclist approaching a roundabout positioned themselves at the edge of the travel lane or in a bike lane or paved shoulder if available. About 15 percent of approaching bicyclists possessed the lane with the remaining 12 percent using the sidewalk. For exiting bicyclists, the percentage on the sidewalk increased to 23 percent and 16 percent possessed the lane. Those bicyclists in the circulatory lane 83 percent of the time, tendered to take the lane rather than ride on the edge of the circle".

Hels and Orozova-Bekkevold (*42*) observed in their research that single bicyclist crashes were under reported since there are usually no conflicting insurance interests. This research also found that single bicyclist crashes often occur in situations where bicyclists 1) collide with the curb or other infrastructure like light posts; 2) ride under the influence and fall over; 3) ride on slippery roads: 4) maneuvers the bicycle poorly, e.g. braking too hard. Also the bicyclist, if seriously injured, often visits the emergency room without contacting the police. Daniels et al. (*43*,*44*) noticed that single lane roundabouts without bicycle paths performed worse than those with bicyclist paths. In addition, they found the construction of roundabouts generally increases the number of severe injury crashes involving bicyclist regardless of the design type of bicycle facilities. Their data showed roundabouts with separated bicyclist paths and grade-separated bicycle paths performed worse when compared to mixed traffic, separate bicycle paths and grade-separated bicycle paths.

Moller and Hels's (*45*) research indicates roundabouts with separated bicycle lanes are safer than roundabouts with mixed traffic or roundabouts with adjacent bicycle lanes. Daniels and Wets (*46*) explained that depending on the design of roundabouts the safety performance for bicyclist could actually worsen. The crash rate for bicyclist in a roundabout with separated bicycle lanes and priority for bicyclists is somewhat higher compared to separated bicycle lanes with no priority for bicyclists (45). Moller and Hels (*47*,35) found that larger central islands appear safer for bicyclists. Inman et al (39), in their research showed that perceived risk is influenced by a combination of factors such as traffic volume, age, gender, and design features regulating the interaction between bicyclists and vehicles. A study conducted in Sweden by Leden et al. (*48*) concluded that the risk to bicyclist and pedestrians decreases with increasing bicyclist and pedestrian traffic volumes.

In summary, roundabouts exhibit an increased safety level because:

- Roundabouts have fewer vehicular conflict points compared with other intersection controls. Potential for angle and head-on crashes is greatly reduced.
- Lower operating speeds allow drivers more time to react to potential conflicts and where there is a crash, the severity is reduced considerable.
- Pedestrians need to cross one direction of traffic at a time at each approach hence reducing the pedestrian-vehicle conflicts.
- Though roundabouts are safer in general than other intersection controls, bicyclist safety is still an issue.

2.5 Performance Measures for Roundabouts

The HCM 2010 specifies LOS criteria for vehicles at roundabouts and is shown in the Table **5**. As the table notes, LOS F is assigned if the volume-to-capacity ratio of a lane exceeds 1.0 regardless of the control delay.

Table 5 LOS Criteria for Vehicles

Control Delay	**LOS by Volume-to-Capacity Ratio**	
(s/veh)	**v/c ≤ 1.0**	**v/c ≥ 1.0**
0-10	A	F
>10-15	B	F
>15-25	C	F
>25-35	D	F
>35-50	E	F
>50	F	F

Source: NCHRP Report 672

Delay data collected in the U.S. suggested that the control delays for roundabouts can be predicted in a manner similar to stop-controlled and signalized intersections (50). Robinson and Rodegerdts (*49*) in their study indicate that 1) degree of saturation, 2) total delay and 3) average queue length are typically used to estimate the operational performance of roundabouts. They explained the need to estimate capacity for a roundabout entry before specific performance measure can be computed. Flannery and Data (*50*) found that roundabouts have great potential for capacity improvement where traffic volumes vary substantially over a period of time. “Roundabouts are known to reduces delays and eliminate the need to stop by replacing the interrupted special and temporal discharge of vehicles on conflicting paths with slow-speed merges and diverges for vehicles moving in the same direction” (32). Generally, if a roundabout is well designed, it can significantly reduce delays at an intersection. Using before and after studies for several intersections, Eisenman et al. (34) found that delay could be reduced by over 50 percent after installing roundabouts. Akcelik (*51*) and Fisk (*52*) identified a number of ways to compute roundabout delay.

For roundabouts, the capacity is evaluated for each approach rather than the intersection as a whole. The HCM 2010 (*53*) explained that the capacity of a roundabout is directly influenced by flow patterns, entering, circulating and exiting vehicles. The capacity of an approach decreases as the conflicting flow increases. Circulating traffic is the primary conflicting traffic stream, though the exiting traffic does affect driver perception during decisions making (*54*). Capacity for a roundabout is not a single value, but a set of values, one for each approach in a time period, and are computed using specific models. Several countries, including U.K., Germany, Australia and France have developed models specifically for their environment. These models generally predict the capacity of a given approach for given conditions using geometric and/or behavioral relationships (32).

The HCM 2010 recommended Equation 1 as the model for estimating the average control delay for each approach lane of a roundabout.

$$d = \frac{3600}{c} + 900T\left[x - 1 + \sqrt{(x-1)^2 + \frac{\left(\frac{3600}{c}\right)x}{450T}}\right] + 5 \times min[x, 1] \qquad (1)$$

Where

d = average control delay (s/veh)

x = volume to capacity ratio of subject lane
c = capacity of subject lane (veh/h)
T = time period (h: T=1 for 1-h analysis, T=0.25 for 15-min analysis)

The 95th percentile queue for a given approach lane is estimated using Equation 2 below

$$Q_{95} = 900T\left[x - 1 + \sqrt{(1-x)^2 + \frac{\left(\frac{3600}{c}\right)x}{150T}}\right]\frac{c}{3600} \qquad (2)$$

Where

Q_{95} = 95th Percentile queue, veh;
x = volume-to-capacity ratio of the subject lane;
c = capacity of subject lane (veh/h)
T = time period (h: T=1 for 1-h analysis, T=0.25 for 15-min analysis)

An exit flow rate greater than 1400 veh/h is unlikely even under good operating conditions hence when exit flows exceeds 1200 veh/h, there may be a need for a double-lane exit (49). Metering is often installed on selected roundabout approaches and operated during periods of heavy demand within the peak hour. This helps approaches with unbalanced flows by reducing unnecessarily long delays and queues (55).

2.5.1 Roundabout Analysis and Design Software

Intersection analysis models can be classified into empirical models and analytical models. Empirical models use observations at many intersections under varying conditions to develop regression equations that match intersection characteristics with intersection capacity and delay. Analytical models estimate capacity based traffic flow theory such as gap-acceptance relationships. The HCM 2010 adopted a combination of a simple lane based regression model and the gap acceptance theory to determine the approach capacity for roundabouts.

Software for roundabout analysis use either macroscopic simulation or microscopic simulation approaches. The microscopic approach is generally implemented in a model that processes individual vehicles and accumulates performance measures based on their progress through the system. Macroscopic models tend to employ flow rate variables and other general descriptors of how the traffic is moving. The flow rate within one segment of the freeway is related to upstream and downstream flow rates through conservation-of-flow equations and other equations that ensure the boundary conditions are met at the interface between system segments.

Most of the popular simulation software in use are grouped under macroscopic or microscopic models and are discussed below. (21,*56*,*57*).

2.5.2 Macro-Simulation Software

The macro-simulation software models commonly used for roundabout analysis include ARCADY, RODEL and SIDRA Solution. RODEL (ROundabout DELay) and ARCADY (Assessment of Roundabout Capacity and DelaY) are empirical macroscopic analysis models for roundabouts that are based on many observations in the United Kingdom. These two programs are often used to estimate the capacities, queues and delays. SIDRA (Signalized and Unsignalized Intersection Design and Research Aid) is an analytical based computer software program developed in Australia for predicting the performance of roundabouts. This analytical model uses an approach based on the gap acceptance theory (also adopted in the HCM) for analyzing non-signalized intersections. The capacity formula calculates the capacity of each approach as a function of the circulating flow, critical gap and follow-up time (56). The HCM software for analysis of roundabout is the Highway Capacity Software (HCS). Some of the popular models are discussed below.

2.5.2.1 RODEL

RODEL is an interactive program intended for evaluation and design of roundabouts. The program was developed in the Highway Department of Staffordshire County Council in England based on an empirical model developed by Kimber (*58*) at the Transport and Road Research Lab (TRRL) in UK. UK chose the empirical model over the gap acceptance model because it directly related capacity to detailed geometric parameters. Required parameter inputs include geometric features such as entry width, approach width, entry radius, and inscribed circle diameter (56). There are two main modes of operation. In mode 1, the user specifies target parameters for average delay, maximum delay, maximum queue, and maximum v/c ratio. RODEL then generates several sets of entry geometrics for each approach based on the given input e.g. width of lane. Depending on site specifics and constraints, the generated geometrics can be used for design purposes. Mode 2 focuses more on performance evaluation using specified values of the geometry and traffic characteristics. RODEL simultaneous displays both input and output data on a single screen which appeals to some users.

2.5.2.2 ARCADY

ARCADY is also a British analysis program with the same empirical theoretical background as RODEL. This software incorporates Kimber's model (58), which used the idea of entry geometry affecting the capacity and related the equation to several site specific parameters. The model assumes a linear relationship between the circulating flow and the maximum entry flow. In ARCADY, input data requirements include entry width, inscribed circle diameter, flare length, approach road width, entry radius, and entry angle. Like RODEL, ARCADY deals in the concept of confidence level. The main difference is that the confidence level may be specified for RODEL, but is embedded in the ARCADY model at 50 percent.

2.5.2.3 SIDRA

SIDRA Solutions was developed by the Australian Road Research Board (ARRB) , as an aid for design and evaluation of signalized intersections, roundabouts, two-way stop control, all-way stop control and yield sign control (*59*). Roundabout capacity estimates are based on the gap acceptance model and are computed separately for each approach lane. This method allows for capacity losses due to lane under-utilization and allocates the largest degree of saturation in any lane movement. In SIDRA, the gap-acceptance parameters are calculated in the following order; The follow up headway in the major traffic flow is estimated as a function of the circulating flow and the inscribed circle diameter; the follow up headway in the minor traffic flow is calculated as a function of the ratio of flows between the lanes considered and the dominant-traffic flow follow-up time. The critical headway is calculated as a function of the follow up headway, the major traffic flow, the number of effective circulating lanes and the entry lane width.

SIDRA requires site-specific data including: 1) traffic volumes by movement, 2) number of entry, exiting and circulating lanes, 3) central island diameter, and 4) circulating roadway width. It uses several parameters for which reasonable default values are offered. One parameter of particular importance is the practical capacity of roundabouts. A default value of 85 percent of the possible capacity (i.e. v/c = 0.85) is used as the maximum operational capacity. SIDRA offers the option to include or exclude geometric delay from computations. The WSDOT (25), MDOT (14), and FDOT (15) use SIDRA as the analysis tool for estimating the capacity performance of roundabouts. FDOT (15) recommended the inclusion of geometric delay since it provides a more realistic assessment of roundabout performance.

2.5.2.4 Highway Capacity Software (HCS)

This is based on the equations developed as part of the NCHRP 3-65 project research on roundabouts and incorporated into the HCM 2010. The HCM 2010 varies slightly from what was introduced in the earlier version. The HCS model is based on the gap acceptance theory. The program allows analysts to assess the operational performance of an existing or planned one-lane or two-lane roundabout based on traffic demand. While the database on which these procedures are based is the most comprehensive developed for U.S. conditions, there are limitations. The limitations include:

- Upstream/downstream signals influence the performance of the roundabout
- Entry “priority reversal” occurrences, such as unusual forced entry conditions under extremely high traffic flows
- A high level of pedestrian or bicycle traffic exists
- Two roundabouts in close proximity
- More than two entry lanes present on one or more approaches.

2.5.3 Micro-Simulation Software

VISSIM and Paramics are two micro-simulation software packages that are popular for roundabout simulation. Unlike macro-simulation models, the user has to write codes to specify details of operations such as entering, circulation, and exiting maneuvers at the roundabout.

2.5.3.1 VISSIM

VISSIM gives a flexible platform that allows the users to realistically model a roundabout using a psycho-physical car following model and a rule-based algorithm for lateral movements. It is based on a link-connector structure which is able to build a complete network or a single intersection. It allows users to import CAD layout (dxf or jpg) and set it as a background on which links can be drawn. An appropriate scale is assigned so that all the measurements are in the same units and all geometric elements are precisely drawn. There are three principal features needed for accurate simulations: 1) approach speed and circulatory speed; 2) priority rules; and finally, 3) traffic assignment. Driver behavior is user defined.

2.5.3.2 PARAMICS

Paramics simulates driver behavior based on a model of the street network and uses gap acceptance theory to determine roundabout operations. This software uses a network (link and node) structure to define the roadway system and an origin-destination matrix to determine vehicle paths through the study area. The output includes both technical data for measurement of effectiveness (e.g. delay) and vehicle animation for visual inspection. It is useful in modeling closely spaced roundabouts since it can account for the interaction between them. Paramics is also good for public involvement because the movement of individual vehicles through a proposed roundabout is clearly illustrated.

2.6 Critical Headway and Follow-Up Headway

Gap-acceptance models are commonly applied for analyzing unsignalized intersections because they capture driver behavior directly and can be made site-specific by customizing the values used for those parameters. However, simple gap-acceptance models might not capture all of the observed behavior, and more complex gap-acceptance models that account for limited priority or reverse priority are difficult to calibrate. Regression models are often used in situations where understanding of driver behavior characteristics is incomplete. The choice of empirical or analytical models for roundabout analysis depends on the agency trails and results obtained since both models produce acceptable outputs depending on prevailing conditions. Based on recent analysis of U.S. field data, "the procedure recommended for use in the U.S. incorporates a combination of simple lane-based regression and gap-acceptance models for both single-lane and double-lane roundabouts" (53). Gap acceptance models require two critical parameters, namely critical headway and follow-up headway, which are defined in the *Highway Capacity Manual* (HCM) as (53):

- *Critical headway.* This is the minimum time between two successive major-stream vehicles in which a minor-street vehicle can make a maneuver (in the case of roundabout critical headway has been historically referred to as critical gap).
- *Follow-up headway*. This is the time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same gap under a condition of continuous queuing.

These two parameters are an indication of the driver behavior at roundabouts and are major factors used to estimate capacity at roundabouts using analytical techniques. These headways are required parameters for calibrating the HCM, 2010 (53) capacity models. Critical headways are also used to calculate intersection sight distance for roundabouts.

2.6.1 Critical Headway and Follow-up Headways Values

The FHWA Guide originally estimated the critical headway value as 6.5 seconds based on the critical headway required for passenger cars assumed to be the design vehicle for intersection sight distance. Some state DOTs that have developed guidelines for roundabout design and analysis use the critical headway recommended by the FHWA Guide. However several states have different values for critical headway. In their supplement to the FHWA Guide, the Kansas Department of Transportation adopts the FHWA Guide's 6.5-second critical headway, but indicates that the critical headway may be reduced to 4.6 seconds in locations where sight distance may be constrained by adjacent topography features or buildings (18). Kansas further recognizes that critical headway should be adjusted to meet the ultimate design objectives, such as the target design speed. The state of Arizona has adopted a similar language.

A recent study for California roundabouts obtained critical headway of 4.8 second for single lane and 4.7 and 4.4 seconds respectively for the left lane and right lane for two lane roundabouts (20). Wisconsin Department of Transportation's roundabout guidance in their *Facilities Design Manual* recommends a critical headway of 4.5 seconds. Based on a study, the Utah Department of Transportation adopted the critical headway values from SIDRA where the minimum critical headway was 2.0 seconds and the maximum critical headway was 8.0 seconds, compared to the two boundaries of the critical headway values from the HCM (4.1 seconds to 4.6 seconds). The HCM critical headway values are mainly used for the purpose of conducting operational analyses. The NCHRP 3-65 report (17) included a set of critical headway and follow-up headway values based on data from more than 500 hours of video at 32 different roundabout locations throughout the U.S. This study (17) obtained critical headways between 4.2 to 5.9 seconds for single lane roundabouts, and 4.2 to 5.5 seconds and 3.4 to 4.9 seconds for the left lane and right lane respectively in the case of double lane roundabouts. In general, most state

DOTs with roundabout guidelines adopted the critical headway recommended by the FHWA Guide.

2.7 Site Selection Guidelines

In general, intersections that meet the criteria for four-way stop control or signal, also qualifies for consideration as a roundabout. At the planning stage, the common considerations that should be addressed are (9):

1) Is a roundabout appropriate for this location?
2) How big should it be or how many lanes might be required?
3) What sort of impacts might be expected? and
4) What public education and outreach might be appropriate?

Prohibitive circumstances that must be considered if a roundabout is to be considered at any intersection are a site where there is insufficient site distance prior to the entrance and the type of design vehicle (*60*). Apart from these, most DOT's list the following situations as suggested conditions for which roundabouts are best suited or used with caution (*14, 15, 22, 23, 25, 30*). These serve only as a guide to preselect the sites before preliminary considerations.

2.7.1 Locations Where Roundabouts are Advantageous

The list identifies conditions where roundabouts can provide advantages over other traffic controls and include:

- Intersections with relatively balanced traffic volumes on each approach
- Intersections where there are a high number of left turn or U-turn movements
- Intersections with safety problems
- "Y" or "T" intersection configuration
- Intersections with large peak period traffic volumes but relatively low traffic volumes during off-peak periods
- Intersections where traffic growth is expected to be high and future traffic patterns are uncertain
- More than four legs or unusual geometry or configuration

- Existing two-way stop-controlled intersections with large side-street delays
- At a gateway or entry point to a campus, neighborhood, or commercial development
- Intersections where widening one or more approach might be difficult or cost-prohibitive, such as at bridge terminals
- Locations where the vehicular operating speed of the road has to be reduced
- Locations with a need to provide a transition between land use types
- Roads with a problem of excessive speeds
- Location with constrained queue storage
- Large traffic signal delays
- Freeway interchange ramp terminals

2.7.2 Locations with Limited Roundabout Opportunities

There are a number of locations and site conditions that often present complications for installing roundabouts. Some of these locations can also be problematic for other intersection control alternatives as well. Therefore, these site conditions should not necessarily preclude a roundabout from consideration. However, extra care should be exercised when considering roundabouts at these locations:

- Intersections in close proximity to a signalized intersection where queues may spill back into the roundabout.
- Intersections located within a coordinated arterial signal system.
- Intersections with a heavy flow of through traffic on the major street opposed by relatively light traffic on the minor street.
- Locations with steep grades or unfavorable topography that might limit visibility and complicate construction.
- Intersections with large bicycle or pedestrian volumes. Some international studies have shown bicyclists might be at more risk at roundabouts than at other intersection types.

2.7.3 Locations Where Roundabouts Might be Inappropriate

Certain locations tend to be disadvantageous for roundabouts. In such situations, other controls should be considered (23, 20). These locations include:

- Places where the cost of right of way is so high that a project becomes uneconomical.

- Where pedestrians regularly comprise the predominant traffic movement through the intersection under present or future conditions (e.g. downtown areas)
- Where grades are significantly greater than 5 percent through the intersection
- Locations with traffic volumes in excess of 50,000 ADT

2.7.4 Roundabouts at Interchanges

Roundabouts can be used within a variety of conventional interchange forms as the means of controlling traffic at ramp terminal intersections experiencing queue blockage and delays, Using roundabouts at interchange terminals is not new and has been successful in the U.K. and other countries (10). Most commonly, roundabouts are used at diamond interchanges but can also be used within partial cloverleaf interchanges (19).

Figure 4 through Figure 6 illustrate how roundabouts are used at interchanges (19). For these interchanges, it is best if the ramp terminal intersections are at least 500 ft apart to avoid the need for widening bridge structures and prevent queues from spilling back into the other intersections. In some cases, the central islands may be raindrop-shaped with no yielding required for traffic between the two roundabouts. If the intersections consist of frontage roads or need to accommodate U-turns, raindrop-shaped central islands should not be used. It must be noted that using roundabouts at interchanges require special considerations e.g. designed to prevent failures and accommodating large trucks.

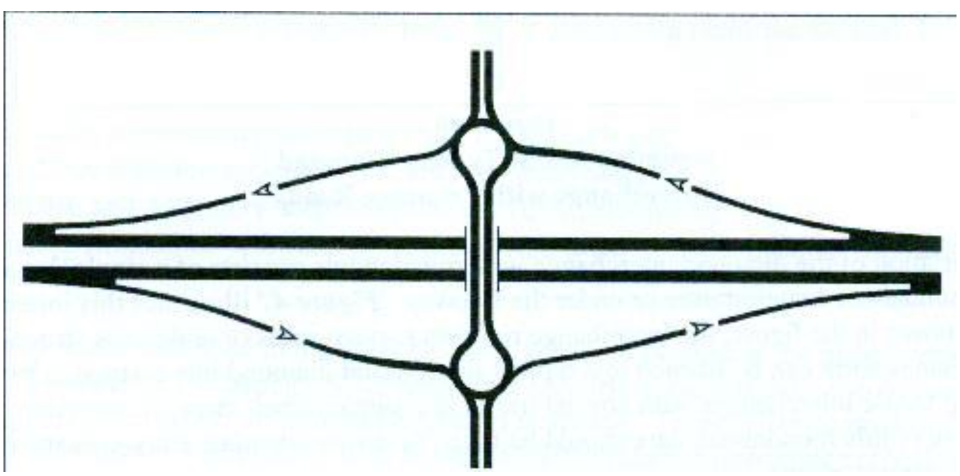


Figure 4: Typical Diamond Interchange with Roundabouts at Ramp Terminal Intersections

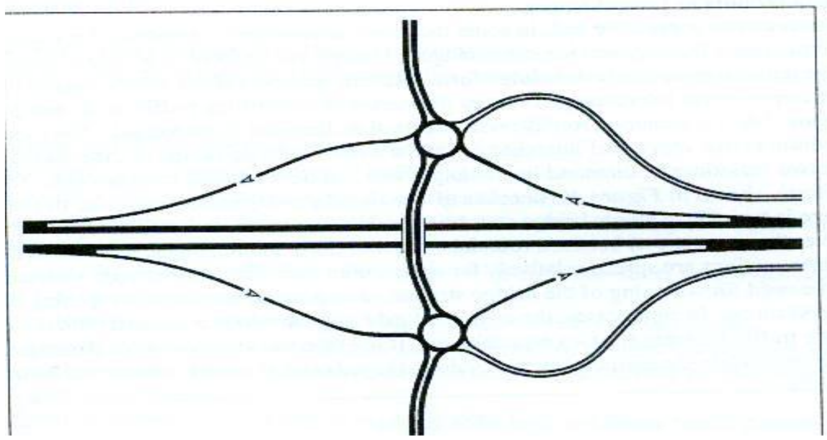


Figure 5 Roundabouts at Typical Diamond Interchange with Frontage Roads

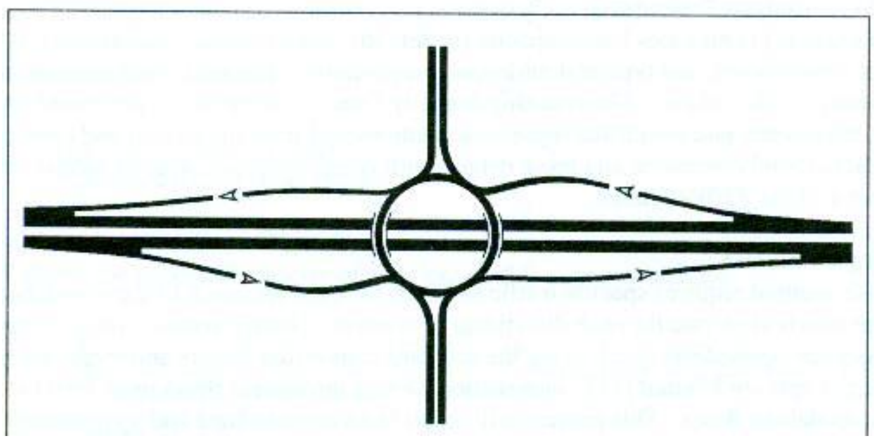


Figure 6: Diamond Interchange with Roundabout at Single Ramp Terminal Intersection

2.8 Roundabout Installation Considerations

Roundabout installation requires consideration of multiple factors. Reviewing several state agency guidelines showed different combinations of factors for roundabout consideration with a summary shown in Table 6.

Table 6 Roundabout Consideration Factors by Agency

Agency	Safety	Cost/ Benefit	Delay/ Queue	Pedestrian/ Bicyclist	ROW	Location/ Site Selection	Environmental Issues	Traffic Vol/ Bal & Trucks	Aesthetics	Capacity	Design Speed	Operational Analysis	Miscellaneous
Maryland	X	X	X	X		X	X	X				X	
Wisconsin	X	X	X	X	X	X	X					X	
Utah	X	X	X	X					X	X			X
Minnesota	X	X		X	X	X		X		X			X
Pennsylvania	X	X		X	X	X	X	X		X	X		
Oregon	X	X	X	X		X				X			
Florida	X	X	X	X			X	X	X		X	X	
Arizona	X			X				X			X		
New York	X			X		X				X			
Washington	X			X						X	X		
Missouri	X		X	X	X	X				X	X	X	
Kansas	X	X	X	X	X			X	X	X		X	
Ada County, Boise	X		X	X	X	X		X		X	X		
Ourston Design	X	X	X	X		X			X			X	
FHWA	X	X	X	X	X		X	X	X			X	X

It was also observed that most state agencies relied on the FHWA's Roundabout Guide (3) for developing their state specific considerations; therefore there was considerable similarity in the rationale for selecting roundabout control for intersections. Burley (30) suggested that the factors usually vary in relative importance from site to site but can be classified broadly into general and site specific factors. General factors include safety, cost, economic evaluation, and community view. Site specific factors are physical controls, road environments, road user costs, and traffic management considerations. Below is a discussion of how the state DOTs applied the factors for roundabout selection.

2.8.1 Safety

Safety is a major reason for most state and local agencies to consider roundabout installation. The MDOT (14), KDOT (18) and Ada County in Idaho (23) recommend the use of roundabouts at locations with high crash rates especially when right-angle and head-on collisions are common. The WisDOT (22) primarily considered roundabouts for intersections with safety problems even though right of way (ROW), cost and operational analysis may normally eliminate them from further considerations. The WSDOT (25) recommended evaluation of safety using collision diagrams, crash types, etc., to evaluate existing conditions, and estimate crash reduction strategies for each feasible intersection control alternative. The PennDOT (29) required five years crash history where available for roundabout consideration. The FDOT (15) required a demonstration of crash reduction with the installation of roundabouts using crash rate and crash type data. The KDOT also required a thorough understanding and evaluation of safety issues before recommending roundabout.

2.8.2 Cost-Benefit Analysis

Cost-benefit analysis for roundabouts includes savings in delay and crashes. Roundabout cost includes construction, relocation of utilities, land acquisition and maintenance which varies widely depending on, location, type and volume of work required. The MDOT (14) required a cost-benefit analysis including 3 years crash data, traffic operations improvements and construction cost. The WisDOT (22) estimated cost based on past typical projects for feasible alternatives and assumes the highest alternative for life cycle evaluation. The PennDOT (29) required a cost based on construction, engineering design, land acquisition and maintenance. The FDOT (15) recommended a comparison with alternate controls to justify roundabouts.

2.8.3 Delay and Queue

Delay and queue evaluations for roundabouts are similar to other intersections controls. Roundabouts generally experience smaller delays compared to other intersection controls with similar traffic volumes lower than 50,000 AADT. The MDOT (14) used the delay and length of queue reduction as justification for roundabout. Ada County, Idaho (23) recommended roundabouts at locations with constrained queue storage and ODOT (16) recommended roundabouts for T intersections and traffic signals with large delays when traffic volume is low.

2.8.4 Pedestrian and Bicyclist

Pedestrians and bicyclist experience greater risk than vehicles at roundabouts therefore they require more protection. The WisDOT recommended identification of nearby pedestrian generators, bike routes and ADA needs to compare the impacts on alternative controls. The MDOT (14) and PennDOT (29) mentioned the need to consider pedestrians and bicyclist in the selection of a roundabout and refer readers to the roundabout guidelines (9). The FDOT (15), KDOT (18) and Ada County (23) rejected roundabouts for sites with dominant pedestrian and/or bicycle traffic volumes either in the present or future.

2.8.5 Right of Way

For built-up areas, the acquisition of additional ROW has critical implications for roundabout cost. For design, the MDOT (14) assumed a right of way requirement comparable to signalized intersections. The KDOT (18) recommended initial sketches done on aerial photographs to estimate ROW needs and acquisition cost. The WisDOT (22) used estimates based on anticipated ROW acreage and real estate cost for each alternative. Ada County (23) rejected roundabouts when the cost of ROW makes it impractical.

2.8.6 Location Selection

Site distance, grade, and general topography affect operations of roundabouts; therefore, most DOT's required in-depth investigations of sites with steep slopes and questionable topography. Ada County (23) recommended roundabouts at intersections with unconventional geometry and further analysis for intersections with grades exceeding 4 percent. The PennDOT (29) recommended roundabouts at intersections that qualify for signals with AADT volumes less than 50,000 since they require similar ROW. The PennDOT's (29) questionnaire used different criteria for new roadways and retrofit roundabouts and also investigated proximity to railway lines, fire stations, etc., that might affect the ROW and operations. The KDOT (18) had different considerations for rural and urban roundabouts because of the distinct driving patterns between the two populations.

2.8.7 Traffic Volume Balance and Truck Percentage

MDOT (14) considered large delay on side streets, flow distribution with heavy left turn movements, and a design hourly volume (DHV) of 7000 vph or vpd or less. The ADOT (19), MDOT (14) and PennDOT (29) recommended that approach traffic flow rates be balanced (approximately equal). The KDOT (18) rejected roundabouts if the side street traffic flow rate is significantly smaller than the main street traffic. PennDOT (29) required percentages of emergency vehicles and trucks as determining factors for ROW requirements and whether an apron was needed. ADOT required the design vehicle turning movement path to be checked for each leg with sketches revised if necessary to ensure good operational performance. The KDOT (18) recommended a selection of the design vehicle based on existing or projected data's largest motorized vehicle to use the intersection. Ada County in Idaho (23) generally recommended roundabouts at intersections with traffic volume that warrants AWSC or traffic signal except when traffic volumes exceed 50,000 AADT. Tee intersections with uncertain future traffic as candidates for roundabouts are recommended by (14, 19, and 23). The FDOT (15) required the peak hour turning movement volume summarized into 15-minute intervals with percentage of trucks as a factor. Most state agencies recommended roundabouts for intersections with high left turn volumes.

2.8.8 Capacity

The PennDOT had a table for determining the preliminary number of lanes of the roundabout depending on traffic volume after capacity evaluation. PennDOT (29) restricted the acceptable volume-to-capacity ratio for all legs to 0.85 based on the FHWA recommendation and further suggested a comparison of the operational capacity of the roundabout with other intersection controls before proceeding to a detailed design. The ADOT (19) recommended roundabouts at intersections with large differences between peak and off-peak period traffic volume demands. The KDOT (18) recommended roundabouts if they have comparable performance to the other control types.

2.8.9 Environmental Issues and Aesthetics

The MDOT (14) suggested roundabouts for intersections with environmental concerns since they produce less noise and air pollution in addition to the possibility of landscaping the central island. The FDOT (15) recommended roundabouts for aesthetic reasons in commercial and civic districts with significantly low traffic volumes that can accommodate pedestrian volumes. The WisDOT (22) required the identification of significant environmental impacts for each alternative control.

2.8.10 Design Speed

A key design requirement of roundabout is the speed reduction features. The ADOT (19) required the difference between design speeds for all legs to be within 12 mph. The FDOT (15) required the same approach speeds on all legs of a roundabout. The FDOT (15), ADOT (19), and Ada County (23) recommended roundabouts for intersections with historical problems of excessive speeds. PennDOT (29) had a table with suggested number of approach lanes for entry design speeds. KDOT (18), FDOT (15), and Ada County (23) recommended roundabouts for intersections requiring speed reduction.

2.8.11 Preliminary Operational Analysis

WisDOT (22) recommended conducting an operational analysis to determine traffic distribution. This was followed by evaluation of geometric improvement needs and conceptual sketches of alternative intersection controls. The ADOT (19) recommended a preliminary operational analysis to determine the number of lanes required before sizing the inscribed circle by encouraged the use of conceptual sketches.

2.8.12 Miscellaneous

Some state agencies recommended special considerations for choosing roundabouts as intersection controls. Examples are proximity to public transit, percentage of older drivers, etc. FDOT (15) and ODOT (16) recommended roundabouts as suitable for sites with unusual geometry or intersections with 5 or more legs.

2.8.13 Summary

In summary, it is clear from this literature that there is insufficient guidance to aid engineers in the determination of intersections that merit roundabouts. Most state agencies discussed general factors suitable for roundabout but did not provide specific guidance as to how these factors interconnect. The WisDOT (22) and MnDOT (28) PennDOT (29) had extensive roundabout selection criteria guide compared with guidelines from other states but these are also inadequate. The FDOT (15) guidelines also gave good indications on roundabout justification.

2.9 Geometric Design Considerations

Geometry design of roundabouts involves several factors including capacity, safety, and delay that need to be satisfied. Other factors involved in geometric design are cost, percentage of trucks and sight distance. The safety and capacity issues are usually at variance with each other. For example, while safety requires geometry that encourages drivers to travel at a safe speed, optimum capacity requires geometry that might encourage drivers to travel above safe speeds. Minor changes in roundabout geometry can create significant changes in capacity, safety and/or operational performances. To achieve optimum design, a tradeoff between the safety and capacity factors is required. Designing a roundabout therefore requires a process of determining the optimal balance between all these factors. Three fundamental elements are required for preliminary roundabout design: 1) optimal roundabout size, 2) optimal location and 3) optimal alignment and arrangement of approach legs *(3)*.

The "Roundabout: An Informational Guide" (3, 9) report was the primary information resource for geometric design information. Other sources of information are guidelines from states that have roundabout programs including, Arizona, California, Florida, Maryland, Oregon, Kansas, Washington, Wisconsin, Utah, Kentucky, Oregon, New York, Pennsylvania, Missouri, Minnesota, Iowa and Idaho. This section discusses the general considerations critical for an effective and efficient geometric design of roundabouts. For the construction phase, further specific design factor will be needed for categories such as double-lane, rural and mini roundabouts. This section addresses the following: 1) lane number and configuration, 2) design vehicle, 3) design speed, 4) speed consistency, 5) inscribed circle diameter, 6) angle between legs 7) intersection sight distance 8) pedestrian and bicyclist considerations and 9) rural roundabouts.

2.9.1 Lane Number and Configuration

Roundabouts are identified in terms of the number of circulating lanes. Determining the number of entering, circulating, and exiting lanes needed for a roundabout are important preliminary factors in the geometric design. Future year design volumes are often recommended for determining the ultimate configuration of roundabouts (3). Since the number of lanes influences capacity, safety, and complexity of operations it is desirable to provide only the required number of lanes though provision can be made for future expansion. Fewer lanes provide less complex maneuvers for motorists, which translates into improved safety. The number of approach lanes required is based on the traffic demand for the approach; consequently, all approaches are not required to have equal number of lanes. However, the circulating roadway needs to adequately accommodate entering lanes; therefore the number of circulating lanes is determined by the number of entering lanes on the respective approaches. Consequently, portions of the circulatory roadway can have more lanes than other portions. The number of exit lanes on a given leg is based on traffic volumes, but never exceeds the number of circulating lanes immediately upstream of the exit. Depending on the peak hour traffic volume and average daily traffic (ADT), a rule-of-thumb for determining the starting point for analysis is presented (29):

a) Use combined entering and circulating volume at merge point;

- For volumes <1100 vph for single-lane approach is sufficient.
- For volumes >1400 vph and <1800 vph a double-lane approach is required.
- For volumes >2300 vph a triple-lane approach is required

b) Using ADT

- If ADT is <22,000 a single-lane roundabout is required.
- If ADT is >27,000 and <39,000 a double-lane roundabout is required.
- If ADT is > 49,000 a triple-lane roundabout is required.

To enable drivers navigate roundabouts as other intersections, it is necessary to maintain consistent lane numbers and arrangements throughout, which minimizes lane changes near or within the roundabout. When vehicles start from the appropriate lane, changes are not required within the roundabout. However, under some circumstances, e.g. roundabouts with more than four legs, it may be necessary to make provision for lane changes in order to satisfy every movement. Some agencies omit circulatory lane striping for some roundabouts because they argue it reduces side swipe collisions for multi-lane roundabouts with lane changing permitted as discussed in NCHRP reports 572 and 672 (9, 17).

Lane numbering for multilane roundabouts is more complex, hence decisions are made on a case by case basis rather than as a general rule. Figure 7 and Figure 8 show examples of roundabout striping approved by the national committee on Manual for Uniform Traffic Control Devices (MUTCD) 2009 illustrating consistency in lane arrangements (*61*). From the two figures, it can be seen that both roundabouts have two entry lanes on all approaches; however, Figure 8 illustrates double left turn movements on two approaches. Multilane roundabouts increase capacity and add conflict points that may prevent them from achieving the same level of crash reduction as single-lane. Figure 9 illustrates additional conflict points on multilane roundabouts. However, even with an expected lower overall crash reduction, multilane roundabouts still result in fewer serious injuries and fatalities when compared to other intersection controls. To determine the appropriate number of lanes, different operational analyses models are available. Examples are the FHWA's Guide simple linear equations, SIDRA, RODEL and VISSIM (3).

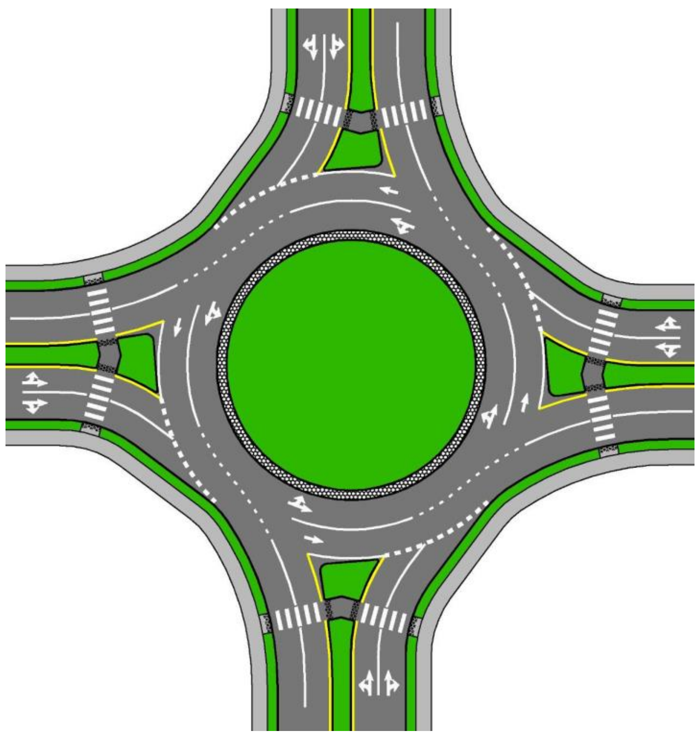


Figure 7: Lane Arrangements for Typical Double-Lane Roundabout

Source: MUTCD 2009

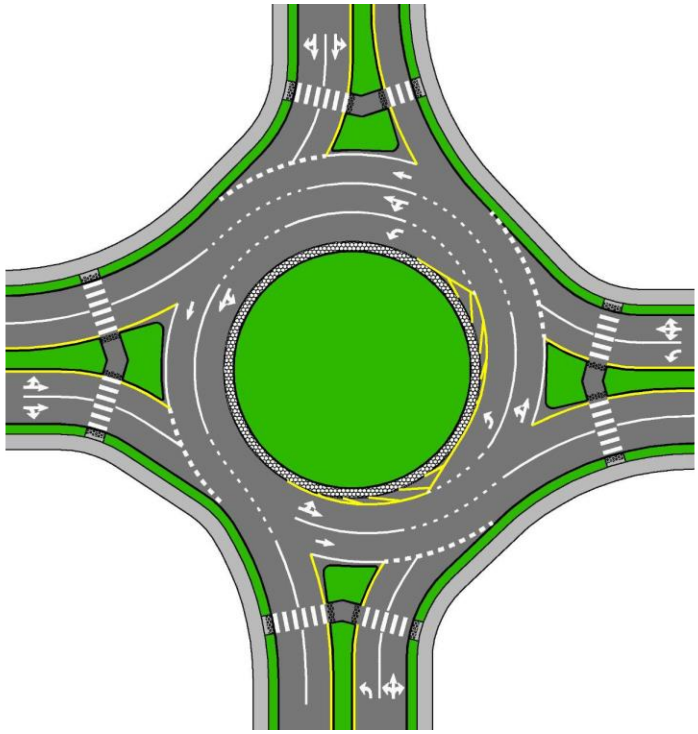


Figure 8: Lane Arrangements for Roundabouts with Consecutive Double-Left Turns
Source: MUTCD 2009

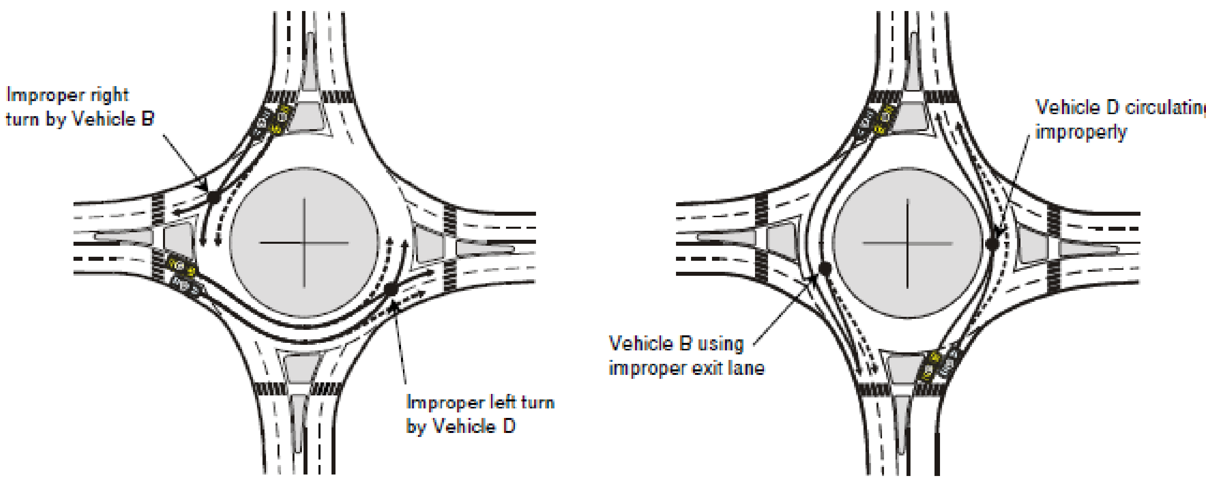


Figure 9: Additional Vehicle Conflicts at Multi-Lane Roundabouts
Source: NCHRP Report 672

2.9.2 Design Vehicle

To start the design process, designers must be conscious of the design vehicle and use appropriate vehicle turning templates or a CAD-based vehicle turning path program to determine the vehicle's swept path (3). The FHWA Guide describes the design vehicle and its turning path requirement which dictates important dimensions for roundabout geometry (3). The choice of a design vehicle depends on the approach roadway and the surrounding land use characteristics. Local and state agencies with roadway jurisdiction should be consulted to select the design vehicle for each roundabout. The FHWA Guide (9), recommends, WB-15 (WB-50) vehicles as the largest vehicles for roundabouts on collectors and arterials and WB-20 (WB-67) vehicles for freeway ramp terminals or state highway systems. For urban areas, bus or single unit trucks could be used while smaller design vehicles are often chosen for roundabouts on local streets (3, 9). In general a balance is required since larger roundabouts are needed to accommodate large vehicles while maintaining low speeds for passenger vehicles. In some locations, ROW constraints might limit the ability to accommodate large semi-trailer combinations while achieving adequate deflection for smaller vehicles. At such locations, a truck apron is useful in providing additional traversable area around the central island for large semi-trailers. Figure 10 and 11 illustrate the swept paths of design vehicles.

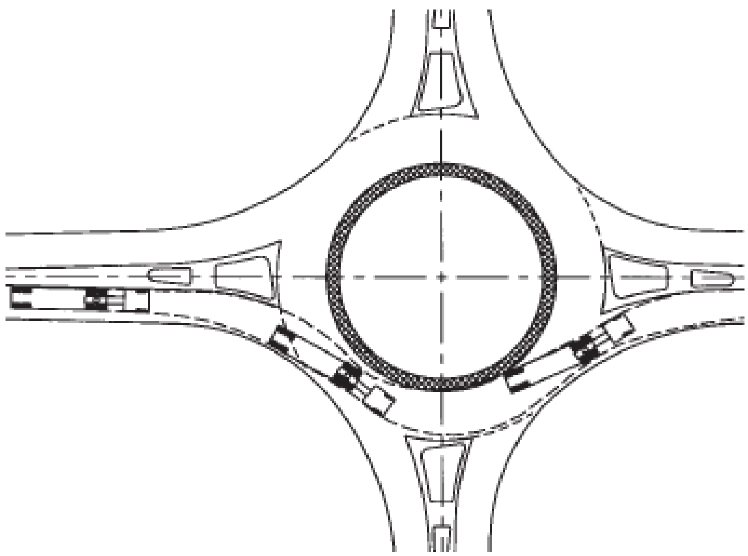


Figure 10: Through-Movement Swept Path of WB-50 (WB-15) Vehicle
Source: FHWA Guideline

The choice of a design vehicle at multilane roundabouts is more complex. Designing roundabouts for two semi-trailer vehicles travelling side by side is very difficult and usually unnecessary. Some combinations of side-by-side vehicles maneuvers (large truck-small vehicle), should however, be accommodated and designed for on multilane roundabouts. This is usually possible on a three-lane circulatory roundabout where the larger vehicle uses two lanes and a standard vehicle uses the adjacent lane. Semi-trailers are usually permitted to track over lane markings within the roundabout entry, circulatory and exit roadways. **Error! Reference source not found.** shows the swept paths of a design vehicle (bus) side by side with a passenger car traversing a multilane roundabout.

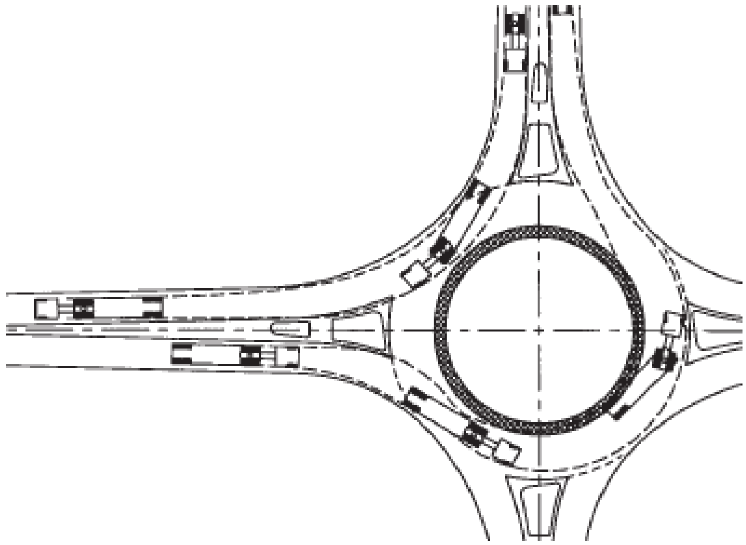


Figure 11: Left-Turn and Right-Turn swept path of WB-50 (WB-15) Vehicle
Source: FHWA Guideline

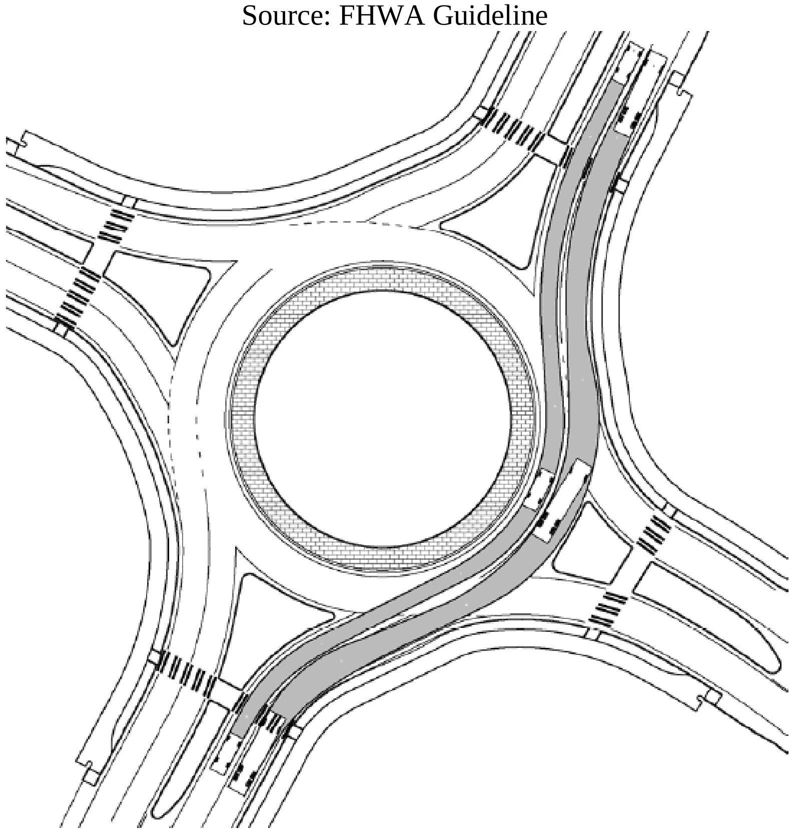


Figure 12: Bus and Passenger Car Swept Paths, Multilane Roundabout

Source: NCHRP Report 672

Table 7 is a summary of some side-by-side vehicle accommodation provisions from different States.

Table 7 Guidance on Multilane Circulatory Roadway Widths

Agency	Source	Inscribed Circle Diameter	Circulatory Roadway Width for WB-67 Design Vehicle	Recommended Pair of Side-by-Side Vehicles for Design
FHWA	*Roundabouts: An Informational Guide (1)*	180 feet	30 feet (minimum)	• Depending on site conditions: • Two passenger cars OR • Passenger car + single-unit truck OR • Semi-trailer + passenger car OR • Semi-trailer + single-unit truck
Kansas DOT	*Kansas Roundabout Guide (2)*	180 feet	30 feet (minimum)	• Depending on site conditions: • Passenger car + bus OR • Passenger car + single-unit truck OR • Semi-trailer + passenger car
Washington State DOT	*WSDOT Design Manual (46)*	N/A	• Maintain 2-ft clearance to any curb face • Minimum circulatory with equal to or slightly wider (120%) than maximum entry width.	N/A
New York State DOT	*Roundabouts: Interim Requirements and Guidance (45)*	N/A	Maintain 3-ft horizontal clearance	N/A
Missouri DOT	*Project Development Manual*	N/A	N/A	Two trucks (type and size not specified)
Wisconsin DOT	*Facilities Development Manual (9)*	N/A	N/A	N/A
Pennsylvania DOT	*Guide to Roundabouts (44)*	N/A	N/A	N/A
Florida DOT	*Florida Roundabout Guide (43)*	N/A	Will not normally exceed 1.2 times the maximum entry width	N/A
Oregon DOT	*Modern Roundabouts for Oregon (41)*	N/A	Will not normally exceed 1.2 times the maximum entry width	N/A
Austroads	*Design Guide For Roundabouts (14)*	180 feet	32 feet	• 1 articulated vehicle + 1 car

Source: California Roundabout Guidelines

2.9.3 Design Speed

Speed at roundabouts is largely recognized as a major factor in the traffic safety. They operate safest when the geometry compels vehicles to enter, circulate and exit at low speeds. The FHWA guide (3) shows that increasing the vehicle path curvature decreases the relative speed between entering and circulating vehicles and results in decreased entering-circulating and exiting-circulating vehicle crashes. At multilane roundabouts on the other hand, increasing vehicle path curvature creates greater side friction between adjacent traffic lanes and can result in more vehicles weaving across lanes and more sideswipe crashes. For every roundabout, there exists an optimum design speed to minimize crashes. The FHWA Guide presents an equation for calculating the design speed for roundabouts and is given by:

$$V = \sqrt{15R(e+f)} \qquad (3)$$

Where

V = Speed (mph)
R = radius (ft)
e = superelevation (ft/ft) and
f = side friction factor

The FHWA Guide recommended common superelevation values of +0.02 for entry and exit curves and -0.02 for circular curves around the central island. The guide also uses a series of graphs to demonstrate the relationship between these parameters (3) recognizing that side friction factors varies with speed. This process can be simplified by fitting an equation to the relationship between speed and path radius for the two common superelevation values. The fitted equations (with a coefficient of determination exceeding 0.997) are as follows:

$V = 3.4415R^{0.3861}$ for e □= 0.02

(4)

$V = 3.4614R^{0.3673}$ for e =□ −0.02 (5)

Where

V = predicted speed (mph)
R = radius of vehicle path (ft)

2.9.3.1 Entry and Exit Design Speed

In the NCHRP report 572 (17), researchers found that if the equations in the FHWA Guide were used the entry and exit speeds were over predicted in cases where the path radius is large and thus they proposed the following equations for estimating the entry and exits design speeds to improve the prediction fit.

Entry Speed

$$V_1 = min\left\{\frac{V_{1phase}}{\frac{1}{1.47}\sqrt{(1.47V_2)^2 + 2a_{12}d_{12}}}\right\} \tag{6}$$

Where

V_1 = entry speed (mph)
V_{1phase} = speed predicted based on path radius (mph)
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
d_{12} = distance along the vehicle path between the point of interest along V_1 path and the midpoint of the V_2 path (ft)

Most states adopted the FHWA guidelines; however, Kansas and Arizona use speeds higher by 5 mph for mini roundabouts, urban compact, and urban single lane roundabouts. Table 8 shows the recommended FHWA speeds compared with those from Kansas and Arizona. The California Guide noted that pedestrian and bicyclist safety and severity of vehicle-vehicle collisions particularly contribute to the selection of a maximum design speed for a roundabout, especially in urban areas.

Table 8 Recommended Maximum Entry Design Speeds

Roundabout Category	Recommended Maximum Entry Design Speed (mph)	
	FHWA	Kansas/Arizona
Mini-Roundabout	15	20
Urban Compact	15	20
Urban Single Lane	20	25
Urban Double Lane	25	25
Rural Single Lane	25	25
Rural Multilane	30	30

Exit Speed

$$v_3 = min\left\{\frac{v_{3phase}}{\frac{1}{1.47}\sqrt{(1.47v_2)^2 + 2a_{23}d_{23}}}\right\} \tag{7}$$

Where

V_3 = exit speed (mph)
V_{3phase} = speed predicted based on path radius (mph)
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
d_{23} = distance between midpoint of V_2 path and point of interest along V_3 path (ft)

As explained in the California Guide (20), this formulation suggests tangential exits do not inherently result in excessive exit speeds as compared to exits with some curvature, provided that circulating speeds are low and the distance to the point of interest on the exit is short (typically the crosswalk). While the authors believe it is desirable to provide some degree of curvature on the exit to reduce the visual appearance of a “straight shot,” such curvature does not appear to always be the controlling factor for exit speeds. In practice, the use of exits with broad curvature or tangential alignments becomes critical for roundabouts with multilane exits. It may be possible, for example, to use a smaller inscribed circle diameter with tangential exits than what might be possible with exits having more curvature. The smaller diameter may result in lower circulating speeds and lower exiting speeds. As with all elements of a roundabout design, the most important principle is integration of all components to achieve a desired result.

2.9.3.2 Vehicle Path

To determine the speed of a roundabout, the FHWA Guide states that, the fastest path allowed by the geometry should be drawn. This is the smoothest and fastest path possible for a single vehicle, in the absence of other traffic and ignoring all lane markings to traverses through the entry, round the central island, and out through the exit. The through movement is usually the fastest possible path, but in some cases a right turn movement might be faster. The design vehicle is assumed to be 2 m (6 ft) wide and maintains a minimum clearance of 0.5 m (2 ft) from a roadway centerline or concrete curb and flushed painted edge line (17). Therefore, the centerline of the vehicle path is drawn with the following distances to the particular geometric features:

- 1.5 m (5 ft) from a concrete curb,
- 1.5 m (5 ft) from a roadway centerline, and
- 1.0 m (3 ft) from a painted edge line

As shown in Figure 13 to Figure 15, the fastest path for the through movement consists of three continuous curves (to the right, to the left, and to the right). The Roundabout Guide, (3, 9) recommend initial drawing of the path freehand. The freehand technique usually provides a more natural representation of the path a driver negotiates in a roundabout, with smooth transitions connecting curves and tangents. Having sketched the fastest path, the designer can then measure the minimum radii using suitable curve templates or by replicating the path in CAD and using it to determine the radii.

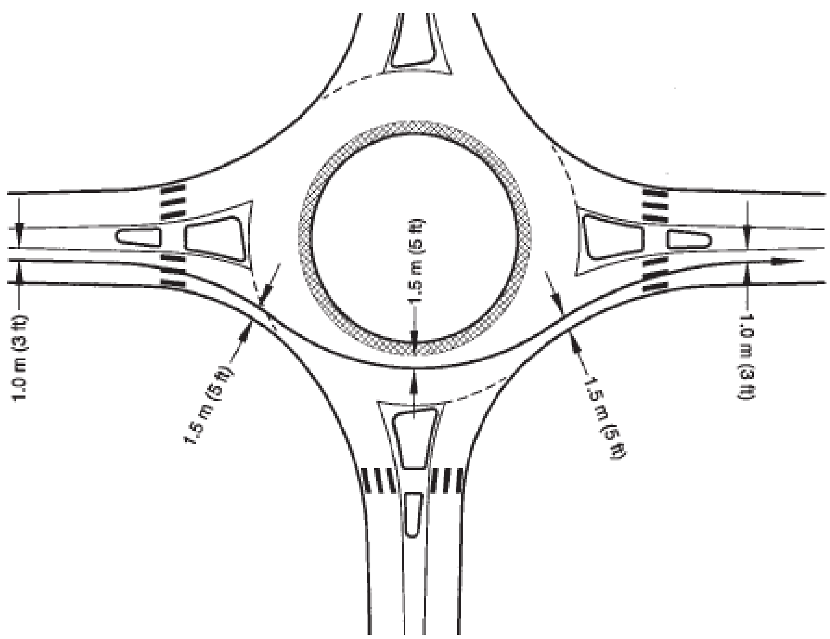


Figure 13: Fastest Vehicle Path through Single-Lane Roundabout
Source: NCHRP Report 672

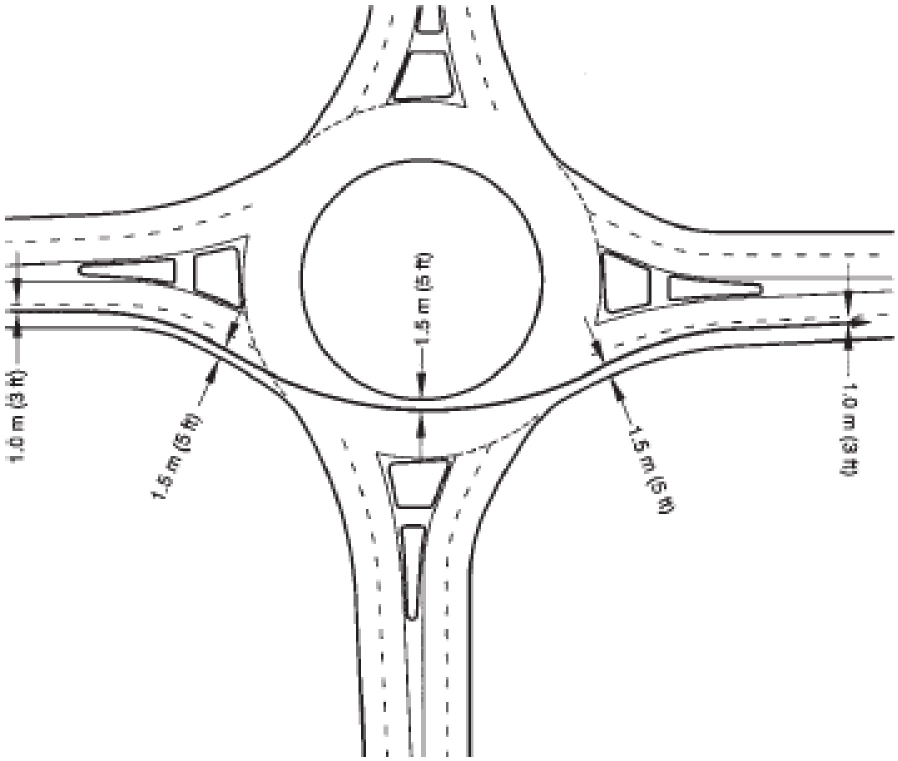


Figure 14 Fastest Vehicle Path through Double-Lane Roundabout
Source: NCHRP Report 672

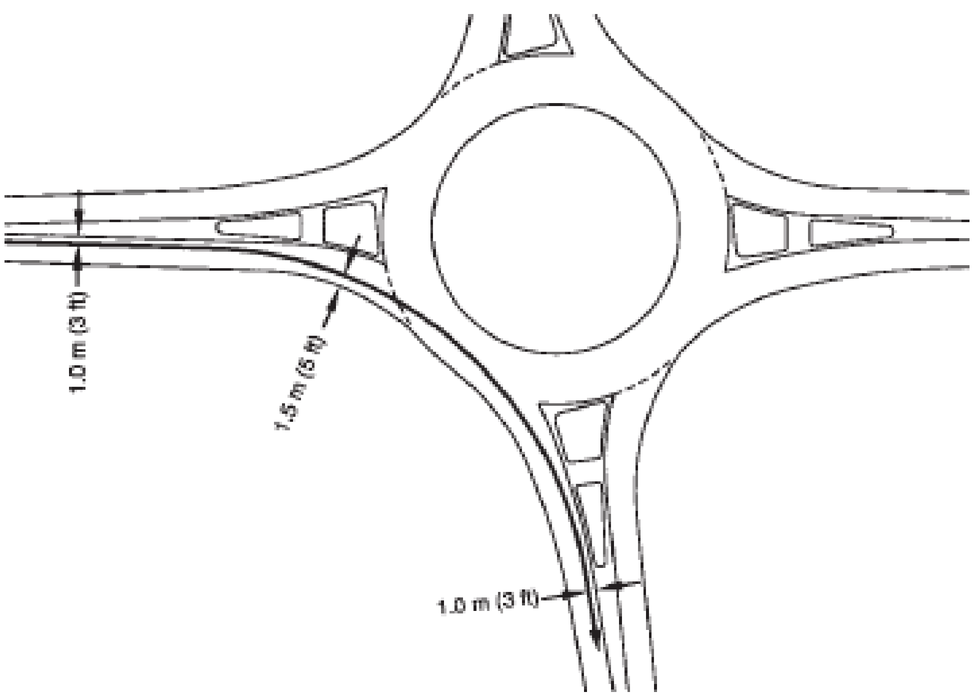


Figure 15: Example of a Critical Right-Turn Movement

Source: NCHRP Report 672

2.9.4 Speed Consistency

Speed consistency was considered by both the FHWA guide (9) and the NCHRP report 572 (17) which concluded that in addition to achieving an appropriate design speed for the fastest movements, it is important also to achieve consistent speeds for all movements. Along with overall reductions in speed, speed consistency can help to minimize total crashes and crash severity rates between conflicting movements of vehicles. It also simplifies the task of merging into the conflicting traffic movement, minimizing critical headways, and optimizing entry capacity (9). Obtaining speed consistency has two implications, that is;

1. Minimize the relative speeds between consecutive geometric elements and
2. Minimize the relative speeds between conflicting traffic movements.

Figure 16 shows the five radii paths that should be checked for each approach to obtain speed consistency. R_1, the entry path radius, is the minimum radius on the fastest through path before crossing the yield line. R_2, the circulating path radius, is the minimum radius on the fastest through path around the central island. R_3, the exit path radius, is the minimum radius on the fastest through path into the exit. R_4, the left-turn path radius, is the minimum radius on the path of the conflicting left-turn movement. R_5, the right-turn path radius, is the minimum radius on the fastest path of a right-turning vehicle. (These vehicular path radii are not the same as the curb radii)

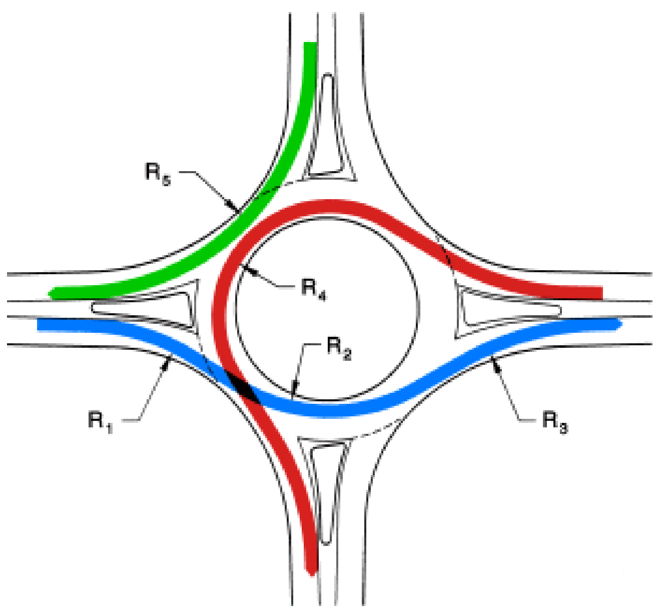


Figure 16: Vehicle Path Radii

Source: NCHRP Report 672

On the fastest path, it is desirable for $R_1 < R_2 < R_3$. This ensures that speeds will be reduced to their lowest level at the roundabout entry and therefore reduce the likelihood of loss-of-control crashes. It also helps to reduce the speed differential between entering and circulating traffic, thus reducing the entering-circulating vehicle crashes. However, in some cases it may be impossible to achieve an R_1 value less than R_2 within given right-of-way or topographic constraints. In such cases, it is acceptable for R_1 to be greater than R_2, provided the relative difference in speeds is less than 12 mph but preferably less than 6 mph (3).

For single-lane roundabouts, it is relatively simple to reduce the value of R_1. At double-lane roundabouts however, it is more difficult as overly small entry curves can cause the natural path of adjacent traffic streams to overlap which may reduce capacity and increase crash risk. An iterative process is thus used to achieve ideal values for R_1, R_2, and R_3, at double-lane roundabouts.

2.9.5 Inscribed Circle Diameter

The inscribed circle diameter is the distance across the circle inscribed by the outer curb or edge of the circulatory roadway. It is one of the critical dimensions affecting the operation of roundabouts. The optimal diameter requires several iterations and depends on the:

1) Turning requirement of the design vehicle
2) Design speed
3) Circulatory roadway width
4) Entry and exit widths
5) Entry and exit radii
6) Entry and exit angles
7) Number of lanes
8) Number of access legs
9) Alignment of legs

The size of roundabouts is usually determined either by the need to achieve deflection or by the need to fit the entries and exits around the circumference with reasonable entry and exit radii between them. Table 9 shows the 2010 FHWA recommended inscribed diameters for different roundabouts giving the design vehicle (9). The guide also suggests that, for double-lane roundabouts, accommodating the design vehicle is usually not a constraint. Some states have a different inscribed diameter recommendation than the earlier FHWA Guide published in 2000 and is shown in Table 10

Table 9 Typical Ranges of Inscribed Diameter for Various Roundabouts

Roundabout Configuration	Typical Design Vehicle	Common Inscribed Circle Diameter Range*	
Mini-Roundabout	SU-30 (SU-9)	45 to 90 ft	(14 to 27 m)
Single-Lane Roundabout	B-40 (B-12)	90 to 150 ft	(27 to 46 m)
	WB-50 (WB-15)	105 to 150 ft	(32 to 46 m)
	WB-67 (WB-20)	130 to 180 ft	(40 to 55 m)
Multilane Roundabout (2 lanes)	WB-50 (WB-15)	150 to 220 ft	(46 to 67 m)
	WB-67 (WB-20)	165 to 220 ft	(50 to 67 m)
Multilane Roundabout (3 lanes)	WB-50 (WB-15)	200 to 250 ft	(61 to 76 m)
	WB-67 (WB-20)	220 to 300 ft	(67 to 91 m)

* Assumes 90° angles between entries and no more than four legs. List of possible design vehicles is not all-inclusive.

Table 10 Typical Inscribed Circle Diameter Ranges

Roundabout Category	Recommended Maximum Entry Design Speed (mph)		
	FHWA (3)	Kansas/ Arizona	Wisconsin
Mini-Roundabout	45-80	50-90	N/A
Urban Compact	80-100	90-120	N/A
Urban Single Lane	100-130	120-150	100-160
Urban Double Lane	150-180	150-220	150-200
Urban Multilane (3 or 4-Lane Entry)	N/A	N/A	180-330
Rural Single Lane	115-130	130-200	115-180
Rural Double Lane	180-200	175-250	180-230
Rural Multilane (3 Lane Entry)	N/A	N/A	180-330

Source: California Department of Transportation Roundabout Guidelines

It is common to see inscribed circles that deviate from the recommendations set out by either the FHWA Guide or the respective states guidelines due to other considerations. An example is designers sometimes use a smaller diameter for a local street/ collector street intersection where the design vehicle may be a bus or single-unit truck. Smaller inscribed diameter roundabouts are generally better for overall safety since they enforce lower speeds but the size selection process is iterative and attempts to satisfy all the necessary factors. It also should be noted that, not all inscribed circles are perfect circles; some are elliptical depending on the approach angle of the legs and the need to satisfy requirements like speed reduction.

2.9.6 Angle between Legs

The NCHRP Report 572 reported that the angle between the legs of a roundabout appears to have a direct influence on entering-circulating crashes (17). The research found that, as the angle to the next leg decreases, the number of entering and circulating crashes increases. Generally, roundabouts are optimally located when the centerlines of all approach legs pass through the center of the inscribed circle making the central island more visible to approaching drivers (3). This also allows the geometry to be adequately designed allowing vehicles to maintain slow speeds through the entries and the exits. In cases where it is impossible to align the legs through the center point, a degree of offset is acceptable. When there are five or more legs or one or more of the legs are skewed especially for multi-lane roundabouts then the spacing between the entries and exits becomes critical to ensure speed reduction. Multilane roundabouts might have another problem when a vehicular path from an entry leg merges with vehicular paths in the circulatory roadway and then diverge at the next exit. This is often a problem when there is wide separation between the entry and exit of adjacent legs as illustrated in Figure 17. The circulatory-exiting conflict shown in Figure 17 can be resolved by realigning two of the legs to achieve an angle of near 90 degrees between all legs as illustrated in Figure 18. Another method of resolving the problem in Figure 17 is by changing the lane striping to force some vehicles into particular paths.

For alignment conflicts, the general solutions are: 1) either to realign one of more of the legs to reduce the angle between the approach legs, 2) modify the lane arrangements by converting the right lane into a right turn only or, 3) convert the left lane into a left turn only lane to reduce conflict points. The modified lane configuration will have to be evaluated for its effectiveness. It is undesirable for an approach alignment to be offset to the right of the roundabout’s center point since this allows vehicles to enter the roundabout at a faster speed resulting in increased “loss of control” and “entering – circulating” crashes. The Roundabout Guide (3) makes recommendation for the approach alignment as illustrated in Figure 19.

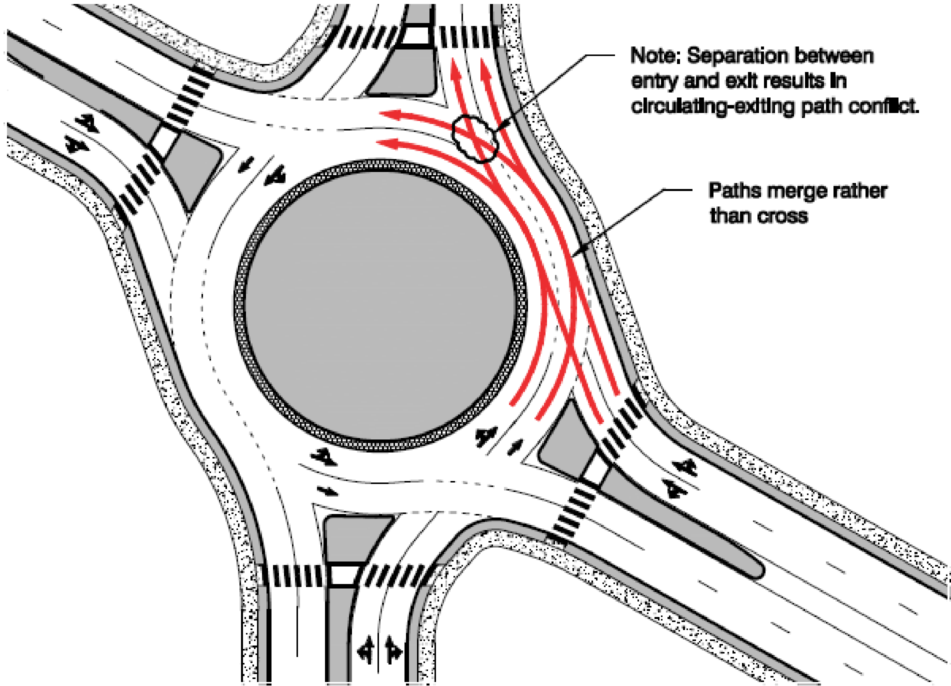


Figure 17: Example of Skewed Legs with Conflicting Circulatory-Exiting Paths
Source: California Roundabout Guidelines

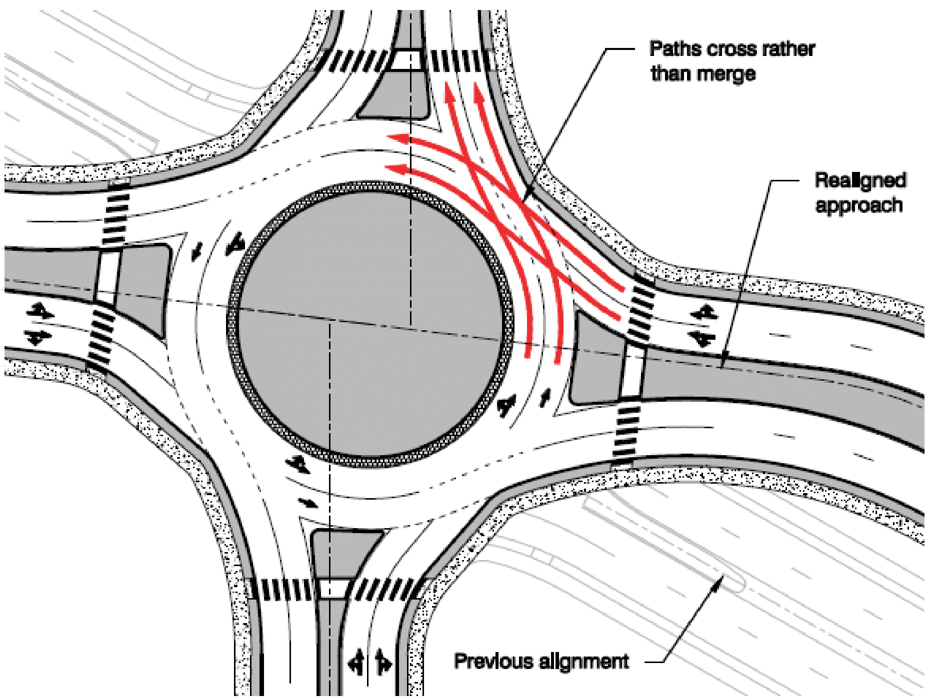


Figure 18: Re-Alignment to Resolve Circulatory-Exiting Conflicts Illustrated in Figure 17
Source: California Roundabout Guidelines

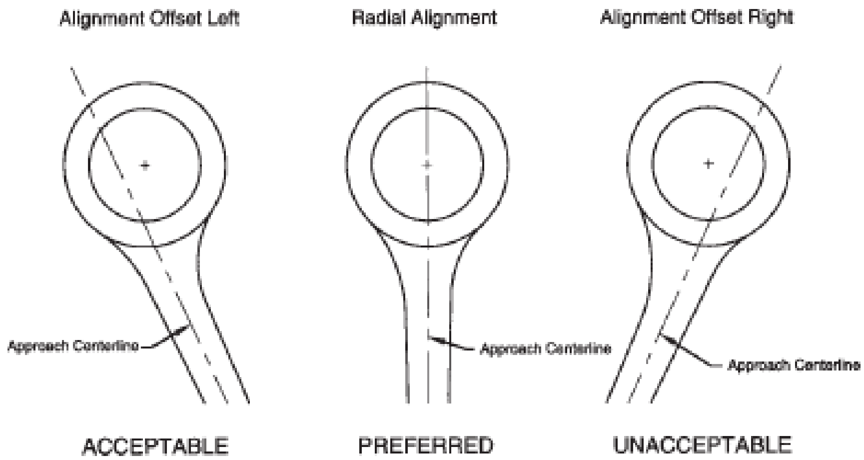


Figure 19: Radial Alignments of Entries
Source: NCHRP Report 672

2.9.7 Roundabout Intersection Sight Distance

This is the distance required for an entering driver to perceive and react to the presence of conflicting vehicles. Adequate intersection sight distance is useful for providing safe operations at roundabouts, but in excess, it might result in higher vehicle entry speeds leading to higher crash frequencies. It is achieved through the establishment of adequate sight lines that allow a driver to see and safely react to potential conflicting vehicles. This provision is required for all entries to a roundabout and is achieved using the intersection sight triangle. For this triangle, two sides are the length of the conflicting roadways that intersect while the third side, which represents the hypotenuse, is the sight distance. The assumption for roundabouts is that the legs follow the curvature of the roadway, and thus distances should not be measured as straight lines but along the vehicular paths. In accordance with the AASHTO "Green Book", the assumed height of driver eye and height of object used are 3.54 ft. The length of the conflicting leg for vehicles approaching a roundabout is computed using the following equation:

$$d_1 = 1.468\left(V_{major\ entering}\right)(t_c) \tag{8}$$

$$d_2 = 1.468\left(V_{major\ circulating}\right)(t_c) \tag{9}$$

Where;

d_1 = length of entering leg of sight triangle, ft
d_2 = length of conflicting leg of sight triangle, ft
V_{major} = design speed of conflicting movement, mph
t_c = critical gap for entering the major road, s, equal to 6.5 s

Note: The critical gap value of 6.5 seconds given above is based on the critical gap required for passenger cars, which are assumed to be the most critical design vehicle for intersection sight distance.

Figure 20 illustrates a typical sight distance determination process with the aid of sketches.

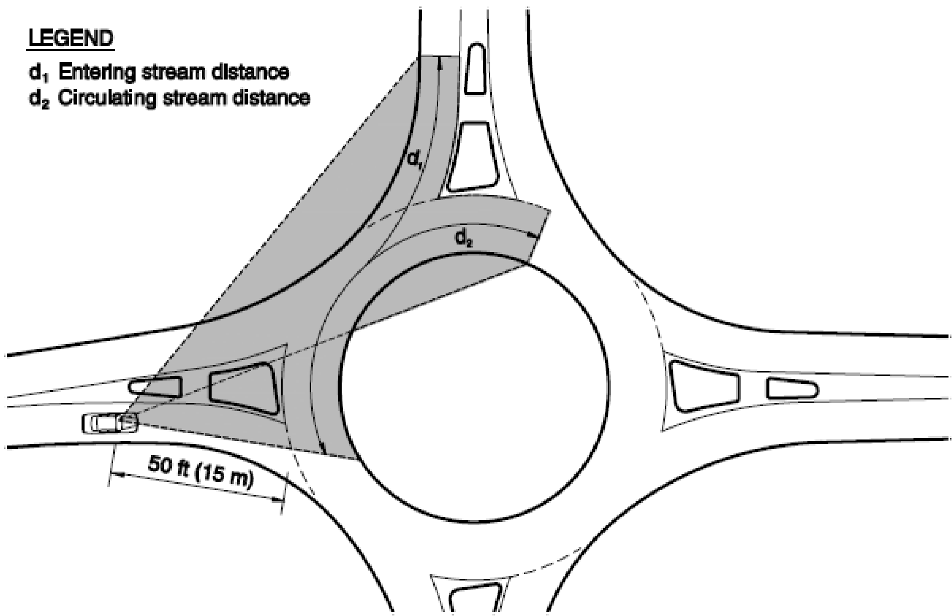


Figure 20: Roundabout Sight Distance
Source: NCHRP Report 672

2.9.8 Stopping Sight Distance

Stopping sight distance is the distance along a roadway required for a driver to perceive and react to an object in the roadway in order to brake and come to a complete stop before reaching that object. The FHWA Guide (9) recommends the provision of stopping sight distance at every point within the circulatory lanes and on the entry and exits. Stopping sight distance is measured using an assumed driver's eyesight height of 3.5 ft and assumed 2 ft for the height of the object in accordance with the AASHTO "green book" (9). The FHWA Guide (9) adopted the equation below for determination of the stopping sight distance of roundabouts

$$d = (1.468)(t)(V) + 1.087\frac{V^2}{a} \qquad (10)$$

Where

d = Stopping sight distance, ft
t = perception-brake reaction time, assumed to be 2.5 second
V = initial speed, mph, and
a = driver deceleration, assumed to be 11.2 ft/s^2

Illustrated in Figure 21 to Figure 23 are three critical locations that should be checked for stopping sight distance: 1) approach, 2) circulatory roadway, and 3) crosswalk at exit.

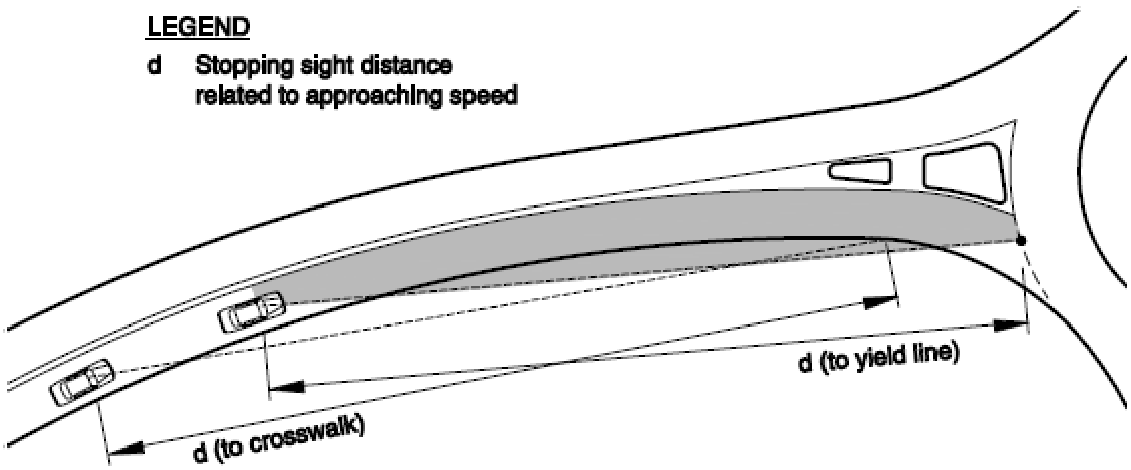


Figure 21: Stopping Sight Distance on the Approach,
Source: NCHRP Report 672

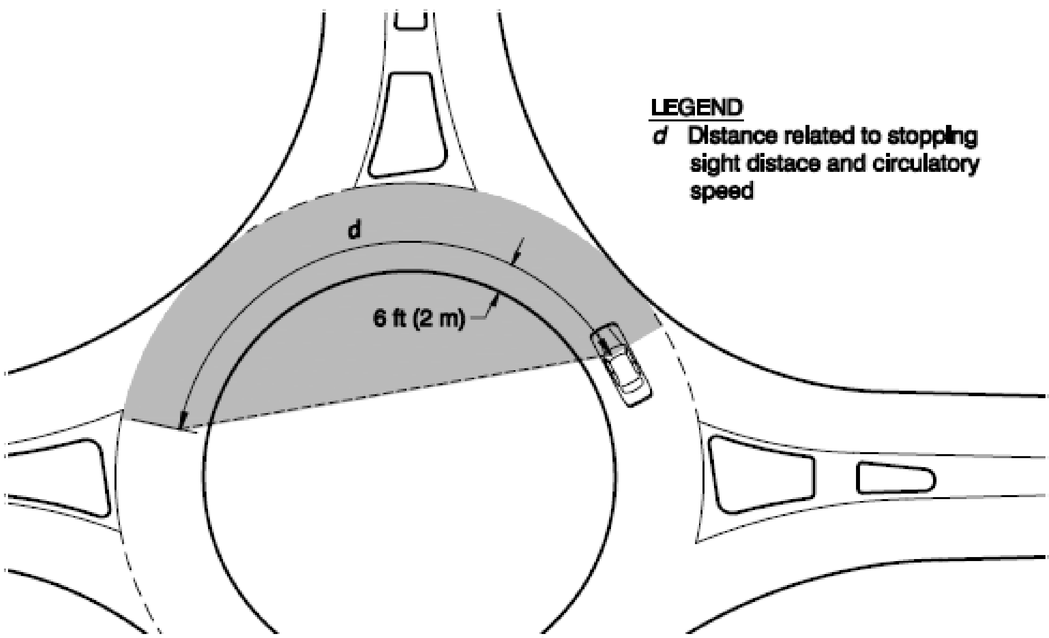


Figure 22: Stopping Sight Distance on Circulatory Roadway
Source: NCHRP Report 672

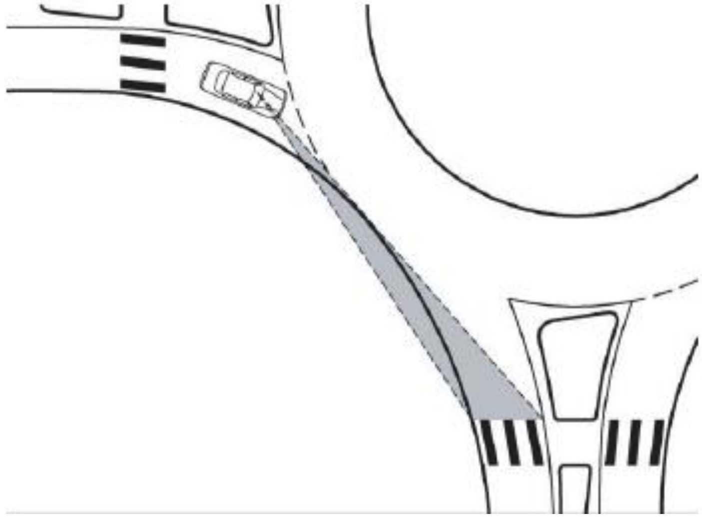


Figure 23: Stopping Sight Distance to Crosswalk on Exit
Source: NCHRP Report 672

2.9.9 Pedestrian and Bicyclist Considerations

Special consideration should be given to non-motorized users of roundabouts. Depending on the activities of these road users, the geometric elements of some components should be adapted to accommodate them and care must be taken as these users span a wide range of age and abilities. The FHWA Guide (3) gives recommendations for the design features that need to be provided for the various categories of non-motorized users and is shown in Table 11.

Table 11 Key Dimensions of Non-Motorized Design Users

User	Dimensions	Affected Roundabout Features
Bicycles		
Length	5.9 ft	Splitter island width at cross-section
Minimum operating width	4.9 ft	Bike lane width
Lateral clearance on each side	2.0 ft to 3.0 ft obstructions	Shared bicycle-pedestrian path width
Pedestrian (Walking)		
Width	1.6 ft	Sidewalk width, crosswalk width
Wheelchair		
Minimum width	2.5 ft	Sidewalk width, crosswalk width
Operating width	3.0 ft	Sidewalk width, crosswalk width
Person pushing stroller		
Effective length	5.6 ft	Splitter island width at crosswalk
Skater		
Typical operating width	6 ft	Sidewalk width

Source: NCHRP Report 672

2.9.10 Rural Roundabouts

Approach speeds for rural roundabouts are usually higher; hence these roundabouts require special design considerations to force drivers to reduce approach speeds. It is necessary to make drivers aware of the roundabout ahead so they can decelerate comfortably to the require speed before attempting to merge. Another way of reducing approach speeds is the use of longer splitter island. A minimum length of 200 ft is recommended.

The FHWA Guide (3) recommends aligning the approach roadways to maximize the visibility of the Central Island and shape of the roundabout where possible. Where it is impossible to provide adequate visibility, additional control devices should be considered for example, advanced warning signs/beacons, pavement markings, , and speed limit reductions. Further enhancements to extended splitter islands include landscaping on the splitter island and road side to create a tunnel effect. Figure 24 is an illustration of a rural roundabout with an extended splitter island.

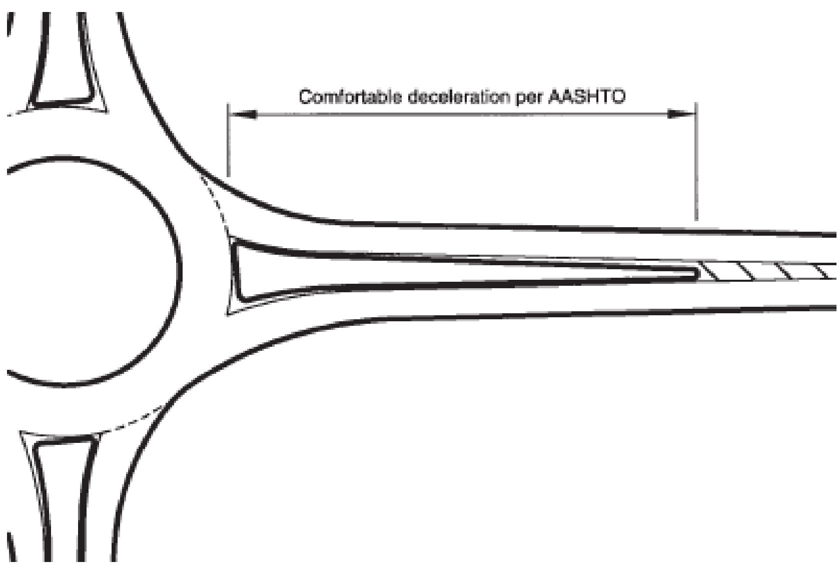


Figure 24: Extended Splitter Island Treatment
Source: NCHRP Report 672

For roundabouts on roads with speed ≥50 mph it is advisable to introduce additional measures to help drivers reduce their speed. One effective method is the introduction of successive reverse curves on the approaches which is illustrated in Figure 25.

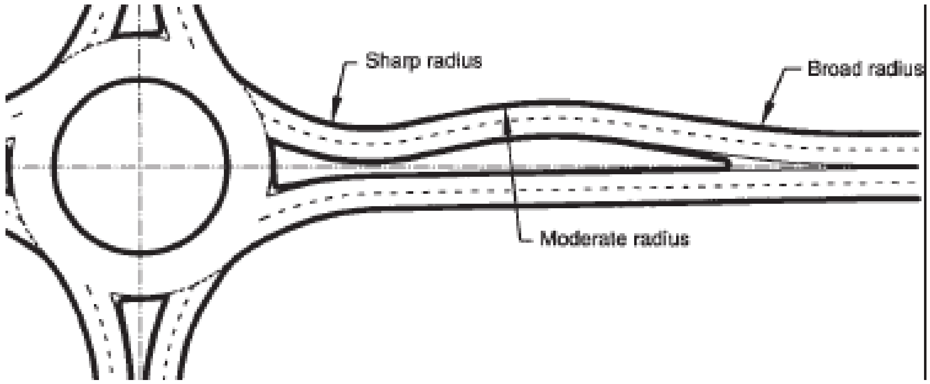


Figure 25: Successive Curves on High Speed Approaches
Source: NCHRP Report 672

2.10 Summary of Findings

The major items discovered in the literature review include the following:

1. Description of roundabout features and categories are adequately addressed in most literature especially the NCHRP Report 672 (3, 9)
2. Safety of roundabouts with regards to vehicles, pedestrians, and bicyclists is well documented. For vehicles-vehicle safety, research has shown reduction in overall crashes and crash severity. Pedestrian safety is also improved for the most part, but there exist perception issues that need to be addressed. Bicyclist's safety however showed mixed reports.
3. Performances are measured for individual approaches rather than the intersection as a whole. Control delay, level of service, and queue length are preferred performance measures.
4. Measured critical headways and follow-up headways vary for different state since they are influenced by driver behavior. It is thus good practice for state DOT's to determine these parameters for capacity estimation and design.
5. There are variations in the factors considered by different states for roundabout site selection. The experience of transportation professionals thus becomes the main tool in the absence of adequate guidance. There is therefore a need to develop a comprehensive guidance for the selection of roundabouts sites.
6. Geometric design guidance is adequately covered in the FHWA guidelines and should serve as a guide to engineers designing roundabouts.

3 Nevada Roundabout Operational Data

3.1 Introduction

The literature review showed that critical headway and follow-up headway are two important parameters required to perform operational analysis and geometric design of roundabouts. These parameters enable the calculations of roundabout capacity, delay, and level of service and also required for calibrating the capacity models in the HCM, 2010 (53).

The first part of the chapter discusses the processes involved in the computation of the mean critical headways and follow-up headways for single-lane and double-lane roundabouts in Nevada. This part describes the field data collection effort which resulted in the measurement of the two parameters, followed by a description of the procedures for obtaining information required for computing the critical headways and follow-up headways. Finally, the Nevada average data was compared with U.S. and foreign average data for statistical significance. The second part describes the calibration of the HCM capacity models using the critical and follow-up headways obtained for Nevada. The third part is the comparison of two roundabout analyses software to determine which one better predicts the performance of Nevada roundabouts.

3.2 Data Collection and Analysis

This section discusses the field data collection, data extraction and the computation of critical headways and follow-up headways.

3.2.1 Data Collection

To obtain data representative of the state of Nevada, the data collection covered the two major metropolitan areas of Nevada: Northern (Reno/Carson) and Southern (Las Vegas). The Northern Nevada data collection involved two single-lane roundabouts: one in Carson City and the other in Fernley. The Southern Nevada data collection involved seven double-lane roundabouts in Clark County. The main criterion for selecting a roundabout site was the presence of sufficient conflicting and entering traffic volumes so that headway data would be obtained for computing the critical headway and follow-up headways. A site reconnaissance always preceded the actual data collection to identify which leg of the roundabout had the highest traffic volume and conflicted by a high volume in the circulatory lane(s).

Data collection for the two Northern Nevada sites was conducted in October 2010 for Carson and June 2011 for Fernley respectively. The data collection for the Southern Nevada sites was conducted in July 2011. To obtain sufficient data, the video recording time at each location for the northern Nevada sites was at least 2 hours. The video recording durations for the southern Nevada sites were at least 2.5 hours since the peak periods generally spanned over 3 hours. All the video recordings were carried out during either the weekday AM peak or PM peak periods when traffic volumes were the highest. Table 12 is a summary of the sites and the associated data collection information.

Table 12 Data Collection Sites and Related Information

No	City	Intersecting Roadways	Data Collection		Number of Entry Lane/ Circulating Lanes
			Date/ Time	Duration of Video Extraction	
1	Carson City	5th St/ Fairview Avenue	10/28/2010 3:45-6:15 pm	2 hrs	1 Lane/ 1 Lane
2	Fernley	US Route 50 Alt/ State Route 343	6/15/2011 4:00-6:30 pm	2 hrs	1 Lane/ 1 Lane
3	Fernley	US Route 50 Alt/ State Route 343	9/19/2011 4:00-6:30 pm	2 hrs	1 Lane/ 1 Lane
4	Henderson	Democracy / Canyon Retreat	7/12/2011 4:00-6:30 pm	2 hrs	1 Lane/ 2 Lane
5	Las Vegas	Town Center Dr/ Village Center Circle	7/12/2011 6:30-9:00 am	2 hrs	2 Lane/ 2 Lane
6	Las Vegas	Town Center Dr/ Haulapai Way	7/11/2011 6:15-8:30 am	2 hrs	2 Lane/ 2 Lane
7	Las Vegas	Town Center Dr/ Banburry Cross Dr	7/11/2011 4:00-6:30 pm	2 hrs	2 Lane/ 2 Lane
8	Las Vegas	Havenwood Lane/ Nevajo Willow Lane	7/13/2011 4:30-6:45 pm	2 hrs	2 Lane/ 2 Lane
9	Las Vegas	Carey/ Rivere	7/14/2011 4:00-6:30 pm	2 hrs	2 Lane/ 2 Lane
10	Las Vegas	Carey/ Hamilton	7/14/2011 6:30-9:00 am	2 hrs	2 Lane/ 2 Lane

The field data collection effort involved recording of vehicle movements at roundabouts using two video cameras mounted on tripods. The geometry, vehicle speeds and any peculiar information judged to have an influence on the headways were noted and recorded. “Camera 1” was used to record the conflicts at the subject entry, while “camera 2” was used to record the traffic movement for the entire intersection. If the entire intersection was captured, the traffic volumes/turning movements extracted from the approaches was to confirm the accuracy of data obtained from "camera 1" and also in the software analysis comparison process discussed later. Figure 26 shows the approximate location of "camera 1". The preferred position for "camera 1" is the point labeled "1a". It was sometimes impossible to use this point either because of the size of the inscribed circle or impossible access to the splitter island. In such instances, point "1b" was the alternative. "Camera 2" was typically positioned outside the immediate inscribed circle with the aim to capture traffic movements on all approaches. If it was impossible because of topography or layout of the roundabout, then a location similar to "1a" or "1b" was chosen but on a different leg to obtain additional data. This data served as supplementary data if needed.

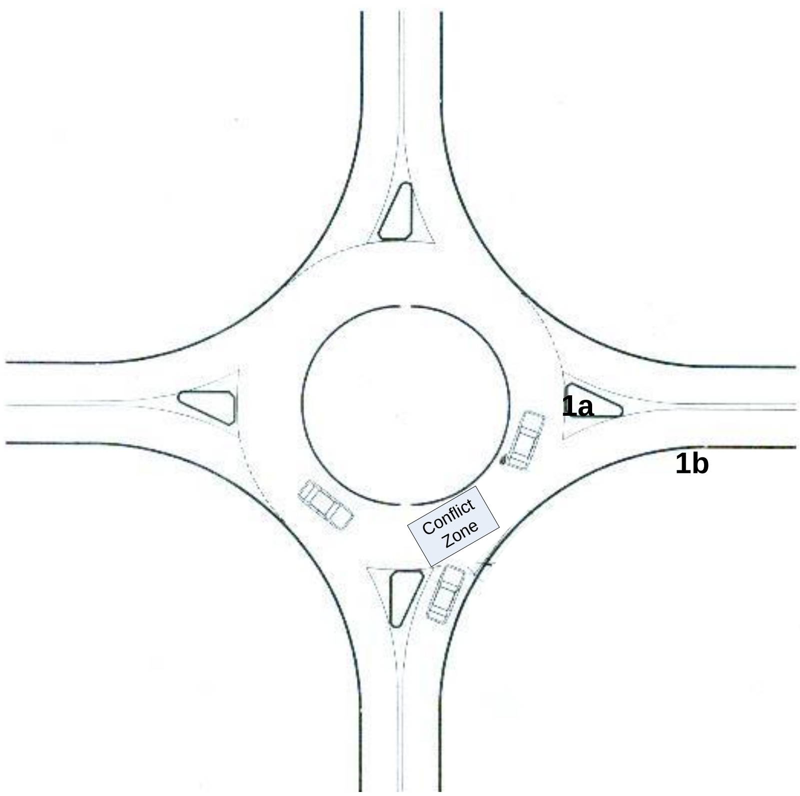


Figure 26: Camera Positions and Conflict Zone

Four observations made during the data collection and worth mentioning are described below:

- *Over-speeding*: For single lane roundabouts, such vehicles occasionally climbed onto the apron to avoid "loss of control". In the case of the multi-lane roundabouts, vehicles usually stride across the two circulating lanes to overcome "loss of control". It is interesting to note that the NDOT crash data reports "loss of control" as one of the top reasons for crashes at roundabouts.

- *Inappropriate use of entry and circulating lanes*: This was observed at multi-lane roundabouts only. It occurs when either a vehicle in the outer approach lane merges onto the inner circulatory lane or a vehicle from the inner approach lane merges onto the outer circulatory lane. Both situations force the circul*ator*y lane vehicles to react.
- *Illegal maneuvers*: two types were observed: 1) drivers approaching and exiting the roundabout on the same leg without going around the central island (U-turn). This particular example occurred at West Carey and Hamilton intersection. A close look at the roundabout revealed insufficient deflection on the approaches as illustrated in Figure 27 which also illustrates the tuning movement *2) Circulatory* vehicles stop or slow down giving way to entering vehicles (priority reversal). These maneuvers are undesirable from the safety and operations standpoints.
- Pedestrians and bicyclists activities were very low at all the roundabout locations and can be attributed to the location being away from commercial centers.

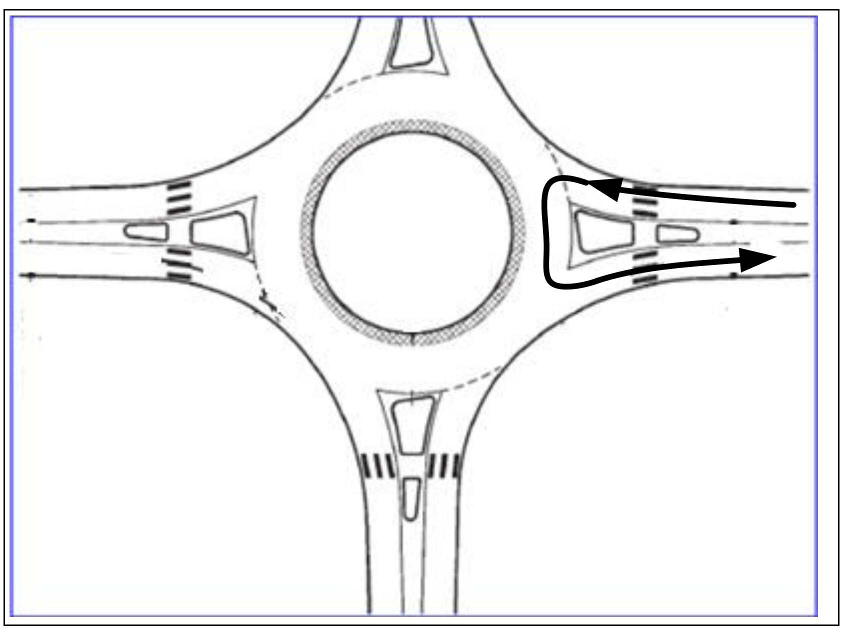


Figure 27: Illustration of an Illegal Maneuver (U-turn)

The roundabout in Fernley operated near saturation volume with long queues but usually discharged quickly. The rest of the roundabouts operated well below their capacities with little or no queues. Three of the double-lane roundabouts were not included in the data analysis because of insufficient headways as a result of low conflicting traffic volume. Circulating vehicle speeds were measured for some of the locations with a handheld speed radar gun for checking the effect of speed on headways.

3.2.2 Data Extraction

Four time events were extracted from the video recordings for the computation of critical headway, follow-up headway and delay. These are described below as:

1. *“Enter queue time”* is the time a vehicle joined the queue. If no queue was observed when the vehicle arrived, a predetermine point on the approach lane was captured as “enter queue time” (a distance approximately 1 to 5 ft from the yield line).
2. *“First in queue” is* the time a vehicle became the first in the queue (at the yield line). This was captured when the vehicle arrived at the yield line. For vehicles that do not make a stop, the event time is when the front bumper of the vehicle just arrived behind the yield line.
3. *“Exit queue time”* is the time a vehicle joined the circulatory lane at the conflict zone (a point on the circulatory lane(s) shown in Figure 26) for vehicles exiting the approach lane(s)
4. *“Passage time” is* the time circulatory lane vehicles arrived at the conflict zone. This defines the headways within the major stream and helps compute the critical headway and follow-up headway.

The “passage times” obtained are used to compute headways between successive circulating vehicles. In combination with the “first in queue times” and “exit queue times’ the “accepted”, “maximum rejected” and the “follow-up” headways were determined. In previous national research e.g. NCHRP 3-46 and NCHRP 3-65, a two-stage approach was used to compute the critical headway and follow-up headway. The first stage involved using a computer software tool to extract the time events from the videos. This computer software, named “Traffic Data Input Program” (TDIP) was developed by the University of Idaho, and helped extract the four time events in two runs. The four time events were manually aligned with the help of Microsoft Excel spreadsheet. The second stage is the computation of the headways which was sometimes done using a Microsoft Excel Macros program.

In this research however, the TDIP was re-coded into "Windows interface program" to reduce the two-stage process described above into a one-stage event. This resulted in time savings and elimination of errors introduced by manually aligning data from the two-stage runs. The new software was developed at the Center for Advance Transportation Education and Research, University of Nevada, Reno and involved three steps:

Step 1: The TDIP (DOS based) computer program was re-programmed using "C-Sharp" computer language into a Windows based interface. The new software enabled all four time events to be extracted in a single run of the video.

Step 2: Coding a Microsoft Excel Macro for the computation of the critical headway and follow-up headway based on Troutbeck's Maximum Likelihood Methodology.

Step 3: Merging Steps 1 and 2 above into a single software which enabled the extraction of the four "time events" and the direct computation of critical and follow-up headways with a button click.

The software used eight seconds as the default upper threshold for the driver acceptable headways. This means any accepted headway larger than 8 seconds was reduced to 8 seconds. This value is consistent with earlier studies carried out in the NCHRP 3-46 and NCHRP 3-65.

Table 13 and Table 14 show the number of accepted and rejected headways for the single-lane and double-lane roundabout sites in Nevada.

Table 13 Accepted and Rejected Headway Cases at Single-Lane Sites

Site	Total No of Headways	Case 1[a]	Percent of Total	Case 2[b]	Percent of Total
Carson	1078	811	75	267	25
Fernley 1	918	487	53	431	47
Fernley 2	619	582	94	37	6
Democracy/Canyon Retreat	99	83	84	16	16
TOTAL	**2714**	**1963**	**72**	**751**	**28**

[a] Case 1: Driver rejected one or more headways
[b] Case 2: Driver accepted the first available headway

Table 14 Accepted and Rejected Headway Cases at Double-Lane Sites

Site		Total No of Headways	Case 1[a]	Percent of Total	Case 2[b]	Percent of Total
North Towne Center/Banburry	LL	381	244	64	137	36
	RL	873	677	78	196	22
North Towne Center/Hualapai	LL	359	287	80	70	19
	RL	524	439	84	85	16
West Carey/Revere	LL	290	213	73	79	27
	RL	622	457	73	165	27
North Towne Center/Village Center	LL	N/A	N/A	N/A	N/A	N/A
	RL	N/A	N/A	N/A	N/A	N/A
West Carey/Hamilton	LL	N/A	N/A	N/A	N/A	N/A
	RL	N/A	N/A	N/A	N/A	N/A
Havenwood/Navajo Wood	LL	N/A	N/A	N/A	N/A	N/A
	RL	N/A	N/A	N/A	N/A	N/A
Total		**3049**	**2317**	**76**	**732**	**24**

[a] Case 1: Driver rejected one or more headways
[b] Case 2: Driver accepted the first available headway
N/A = Not Applicable

3.3 Observed Critical Headway and Follow-Up Headway

Critical headway and follow-up headway are affected by the traffic circulation flow pattern and road geometric features like number of circulating lanes and average width of an entry lane at the intersection (*62*). For the NCHRP 3-65 project and other similar projects, Rod Troutbeck's Maximum Likelihood Methodology (*63*) was used for the computation of the critical and follow-up headways. The "Maximum Likelihood Methodology" is based on the assumption that the minor stream drivers behave consistently, meaning, every driver has a certain critical headway that is acceptable (53). Using the Maximum Likelihood Methodology to calculate critical headway (t_c) relies on information from the number of rejected and accepted headways, therefore, it is difficult to directly measure directly in the field. One key component of the method is that it estimates the average critical headway of all the drivers based on the principle that a driver's critical headway is between the driver's largest rejected headway and the accepted headway that is observed. Unlike the critical headway, the average and standard deviation of the follow-up headways (t_f) are computed directly from the observed values. This is because from the definition, it is described as the average time gap between two cars of the entering stream being queued and entering the same mainstream gap one behind the other.

Measuring the critical and follow-up headways for double-lane and other multilane roundabouts are defined differently. For example in double-lane roundabouts, whereas the right-lane is considered to conflict only with vehicles in the right-lane of the circulatory lanes, the left-lane is considered to conflict with both circulatory lanes. The data extraction is therefore based on the number of conflicting lane(s) for the subject approach lane.

3.3.1 Single-Lane Roundabouts

Table 15 is a summary of the critical headways and follow-up headways obtained for single-lane roundabouts. The critical headway values for single entry lane conflicted by single circulatory lane ranged between 3.3 and 4.4 seconds and the follow-up headway ranged between 2.7 and 3.3 seconds. For the single entry lane conflicted by 2 circulatory lanes the critical headway is 3.1 seconds. The follow-up headway was not measurable because no queues were formed.

Table 15 Critical Headways and Follow-up Headways at Single-Lane Roundabout Sites

Site	Mean of Headways	Standard Deviation of Headways	Follow-up Headways	Standard Deviation of Headways	No of Entry Lane/Circulatory Lanes
Carson	4.4	1.8	3.3	0.7	1 lane/ 1 lane
Fernley 1	4.0	1.5	2.8	0.2	1 lane/ 1 lane
Fernley 2	3.3	2.0	2.7	0.3	1 lane/ 1 lane
Dem/CR	3.1	1.0	N/A	N/A	1 lane/ 2 lane
Average	3.7	1.6	2.9	0.4	1 lane/ 1 lane

3.3.2 Double-Lane Roundabouts

Table 16 is a summary of the critical headways and follow-up headways for double-lane roundabouts. The right-lane critical headways ranges between 4.3 and 5.5 seconds with a mean of 4.8 seconds, and the left-lane critical headways ranged between 4.1 and 5.9 seconds with a mean of 4.9 seconds. The right-lane follow-up headway ranged between 2.6 and 3.2 seconds with an average of 2.9 seconds and the left-lane follow-up headway ranges between 2.3 and 3.5 seconds with an average of 2.9 seconds.

Table 16 Critical Headways and Follow-up Headways at Double-Lane Roundabout Sites

Site		Mean of Headways	Standard Deviation of Headways	Follow-up Headways	Standard Deviation of Headways
North Towne Center/ Banburry	LL	5.9	2.7	3.5	1.0
	RL	5.5	1.9	3.2	0.5
North Towne Center / Hualapai	LL	4.1	0.9	3.0	0.7
	RL	4.5	2.0	3.1	0.2
West Carey/ Revere	LL	4.6	1.6	2.3	0.2
	RL	4.3	1.9	2.6	0.5
Average	LL	4.9	1.8	2.9	0.7
	RL	4.8	1.9	2.9	0.4

3.4 Comparison with National and Foreign Data

To verify the Nevada data, comparisons were made with data from other sources including the NCHRP 3-65 project, Germany, France, Australia, HCM and California State Roundabout Guidelines. The NCHRP 3-65 project consisted of data from several states including Maryland, Vermont, Washington, Maine, Michigan and Oregon. Table 17 summarizes the critical headways and follow-up headways from these sources.

Table 17 Critical Headways and Follow-up Headways from Different sources

Model		Critical Headway		Follow-up Headway	
		1 Lane	2 Lane	1 Lane	2 Lane
HCM		4.1 - 4.6	N/A	2.6 - 3.1	N/A
Germany[1]		4.1	4.3	2.9	2.5
France[2]		N/A	N/A	2.1	2.1
Australia[3]		3.45	3.45	2.04	2.04
NCHRP 3-65[4]	Left Lane	4.2 – 5.9 (5.1)	4.2 - 5.5 (4.5)	2.6 - 4.3 (3.2)	3.1 - 4.7 (3.4)
	Right Lane		3.4 - 4.9 (4.2)		2.7 - 4.4 (3.1)
California[5]	Left Lane	4.5 – 5.3 (4.8)	4.4 -5.1 (4.7)	2.3 – 2.8 (2.5)	1.8 – 2.7 (2.2)
	Right Lane		4.0 – 4.8 (4.4)		2.1 – 2.3 (2.2)
Nevada	Left Lane	3.3 - 4.4 (3.9)	4.1 - 5.9 (4.9)	2.7 - 3.3 (2.9)	2.3 - 3.5 (2.9)

	Right Lane		4.3 - 5.5 (4.8)		2.6 - 3.2 (2.9)

1 Results obtained from NCHRP Report 572
2 Results obtained from Brilon, W. (*64*)
3 Results obtained from Akcelik, R., and Besley, M., (*62*)
4 Results obtained from Rodegerdts et al (17)
5 Results obtained from Tian et al (20)
N/A = Not Applicable
The values in () are the average

From the table, it is observed that whereas the average critical headway for Nevada single lane roundabouts was lower than the NCHRP 3-65 results; the critical headways for double-lane roundabout were higher than the NCHRP 3-65 average in both lanes. However, the mean follow-up headways for both the single-lane and double-lane (both lanes) roundabouts in Nevada were lower than the NCHRP 3-65 project results.

To verify if the Nevada data were statistically different or otherwise from those computed for other states in the U.S., statistical analyses were conducted to compare the Nevada drivers' critical and follow-up headways to data from the NCHRP 3-65 project. Two statistical methods were used for the comparison: "confidence interval hypothesis testing" and "two samples t-test". The two methods are used for testing difference between the means of small sample sizes. For the confidence interval hypothesis testing, if the confidence intervals plots of the populations being compared overlapped, it implied there was no significant statistical difference between the means. For the two sample t-test, if the confidence intervals range included zero (0) then, there was no statistical difference. Both statistic tests were conducted using MINITAB computer software at a 95 percent confidence interval.

3.4.1 Critical Headways

Figure 28 shows the comparison of critical headways for single-lane roundabout sites. The NCHRP 3-65 plot had 18 data points from four states: Washington, Maryland, Maine and Oregon. The 95 percent confidence intervals for the Nevada data was (2.48, 5.30) and the 95 percent confidence interval for the NCHRP data was (4.89, 5.36). The confidence intervals from the two data sources overlapped, meaning statistically the two populations were not significantly different at a 5 percent significance level. However, a look at the plots shows that the mean of the Nevada data is much lower than the mean for the NCHRP data. The two-sample t-test was also used to compare the mean critical headways obtained for single-lane roundabouts sites. The comparison gave the 95 percent confidence interval to be (-2.732, 0.278), the P-value = 0.073, the T-value = -3.51 and degree of freedom (DF) 2. This can be interpreted to mean the null hypothesis of $\mu_1=\mu_2$ is not rejected, which confirms that there was no significant statistical difference between the two data sets.

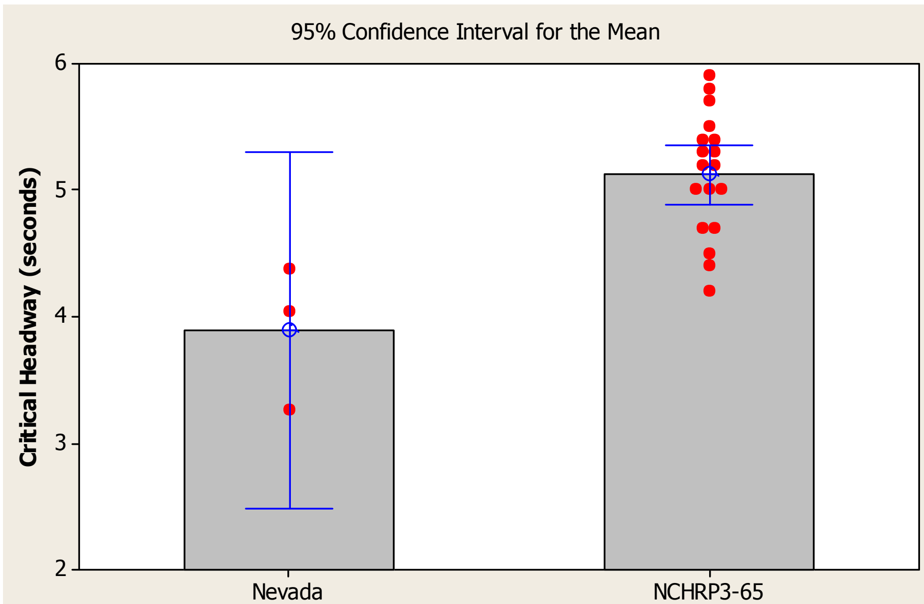


Figure 28: Comparison of Critical Headways from Nevada and NCHRP 3-65 (Single-Lane Sites)

Figure 29 shows the comparison of critical headways at double-lane roundabout sites; compared separately for the left lanes and the right lanes. The NCHRP 3-65 plots consisted of seven sites each from three states: Maryland, Vermont and Washington. Comparing the 95 percent confidence interval plots in Figure 29, it can be concluded that, there was no significant statistical difference between the two data sources for both lanes. From the plots, it is also noted that the mean critical headways from the left lanes were slightly higher than the right lane. This is consistent with field expectations since the right lane is considered to only conflict with one circulating lane compared to two conflicting lanes for the left lane. Examining the plots showed that the means for the left lanes are fairly close compared to those of the right lanes.

The two-sample t-test was again used to compare the difference in the mean critical headways obtained for double-lane roundabout sites. The comparison resulted in a 95 percent confidence interval of (-2.137, 2.756) for the left lane and (-1.30, 2.530) for the right lane. The left and right lanes had P-values of 0.641 and 0.066; T-values of 0.54 and 2.50; and DFs of 2 and 4 respectively. This results lead to the conclusion that the null hypothesis of $\mu_1=\mu_2$ is not rejected for both lane. This confirms the earlier interpretation that there was no significant statistical difference between the two data sets.

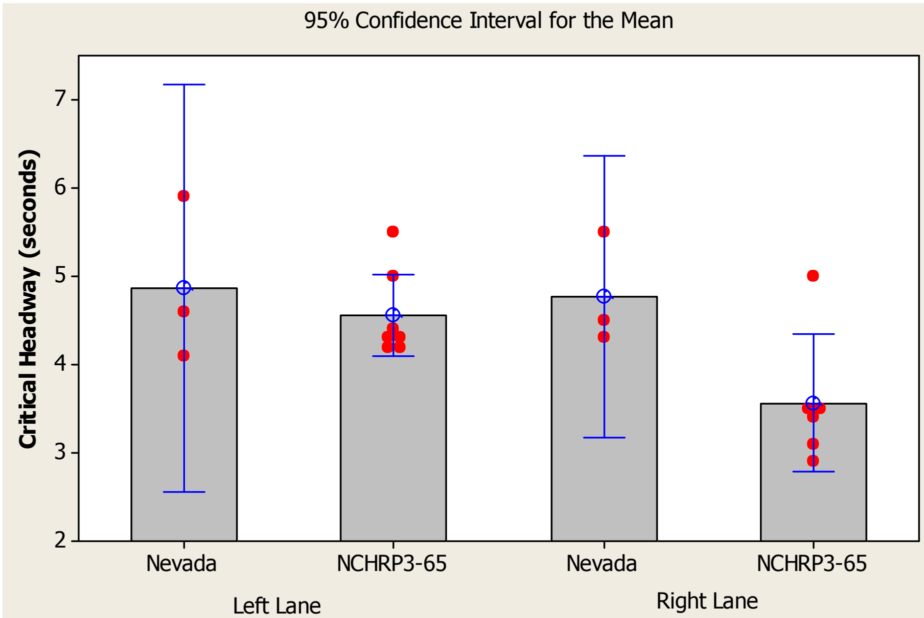


Figure 29: Comparison of Critical Headways from Nevada and NCHRP 3-65 (Double-Lane Sites)

3.4.2 Follow-Up Headways

Figure 30 shows the comparison of follow-up headways of single-lane roundabout sites. The figure shows that the 95 percent confidence interval plots overlapped indicating there was no significant statistical difference between the two data sets at the 5 percent significance level. A look at the plots showed that the mean of the Nevada data was lower than the mean for the NCHRP data. Using the two-sample t-test to compare the difference in follow-up headways obtained for single-lane roundabout sites, the 95 percent confidence interval was (-1.289, 0.557), P-value of 0.23, the T-value of -1.71 with DF 2. This result can be interpreted to mean the null hypothesis of $\mu_1=\mu_2$ is not rejected, confirming the earlier results.

Figure 31 shows the comparison of the mean follow-up headways for double-lane roundabout sites; compared separately for the two lanes. The 95 percent confidence interval plots overlapped for both lane comparisons. This is interpreted to mean that, there was no significant statistical difference between the follow-up headways for Nevada and the NCHRP 3-65 project data at a 5 percent significance level. Despite the overlap, a look at the plots show that the mean for the right lanes were much closer compared the means for the left lanes.

The two-sample t-test was used to compare follow-up headways for double-lane roundabout sites in Nevada and the NCHRP 3-65 project. The results from the comparison gave the 95 percent confidence intervals to be (-2.512, 0.046) for left lanes and (-1.096, 0.363) for right lanes. The left and right lanes had P-values of 0.055 and 0.265; T-values of – 3.07 and -1.23; and DFs of 3 and 6 respectively. This results lead to the conclusion that the null hypothesis of μ_1=μ_2 is not rejected for both lanes confirming the results obtained earlier.

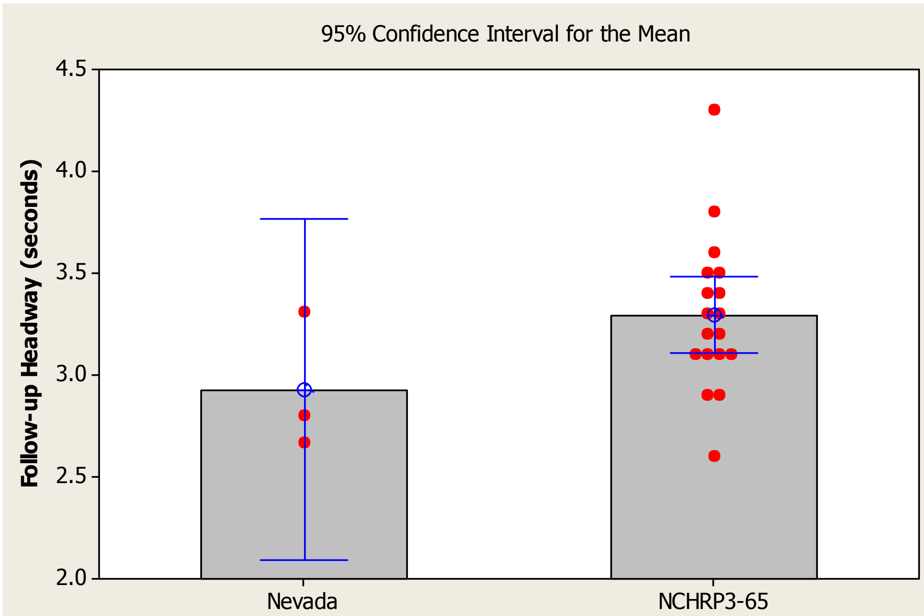


Figure 30: Comparison of Follow-up Headways from Nevada and NCHRP 3-65 (Single-Lane Sites)

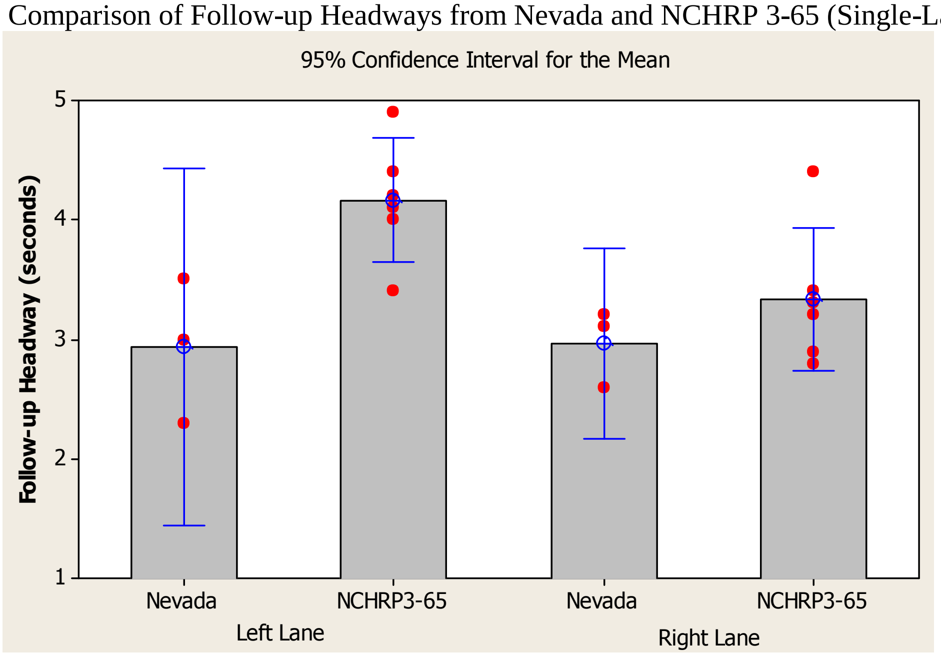


Figure 31 Comparison of Follow-up Headways from Nevada and NCHRP 3-65 (Double-Lane Sites)

It must be noted that for both roundabout types, the Nevada plots consisted of only 3 data points hence the 95 percent confidence interval plots had a wide range; in the presence of more data the mean headways might change resulting in different interpretations.

In summary it can be concluded from the two statistical analyses that at a 5 percent significance level, the mean critical and follow-up headways for both single-lane and double-lane roundabouts in Nevada are not significantly different from the NCHRP sites. However, the mean headways obtained differed from those used for the HCM, 2010 models, therefore, there is the need to calibrate the capacity equations to reflect the Nevada situation.

3.5 Calibration of Capacity Models

The HCM 2010 provides for individual states to calibrate the capacity models if the critical headway and follow-up headway vary from those used. The capacity equations for various entry lanes in the HCM 2010 are given as;

For single-lane entry conflicted by one circulating lane

$$C_{pce} = 1{,}130e^{(-1.0\times10^{-3})v_{c,pce}} \tag{11}$$

For double-lanes entries conflicted by one circulating lane

$$C_{pce} = 1{,}130e^{(-1.0\times10^{-3})v_{c,pce}} \tag{12}$$

For single-lane entry conflicted by two circulating lanes

$$C_{pce} = 1{,}130e^{(-0.7\times10^{-3})v_{c,pce}} \tag{13}$$

For double-lane entries conflicted by two circulating lanes

$$C_{e,R,pce} = 1{,}130e^{(-0.7\times10^{-3})v_{c,pce}} \tag{14}$$

$$C_{e,L,pce} = 1{,}130e^{(-0.75\times10^{-3})v_{c,pce}} \tag{15}$$

Where;

C_{pce} = lane capacity, adjusted for heavy vehicles, pc/h;

$C_{e,R,pce}$ = capacity of right entry lane, adjusted for heavy vehicles, pc/h;

$C_{e,L,pce}$ = capacity of left entry lane, adjusted for heavy vehicles, pc/h; and
$v_{c,pce}$ = conflicting flow pc/h.

To calibrate the capacity equations using the headways observed for Nevada, Equations 16 to 18 given in the HCM, 2010 (53) are used:

$$C_{pce} = Ae^{(-Bv_c)} \tag{16}$$

$$A = \frac{3,600}{t_f} \tag{17}$$

$$B = \frac{t_c - (t_f/2)}{3,600} \tag{18}$$

Where;
C_{pce} = lane capacity, adjusted for heavy vehicles, pc/h;
v_c = conflicting flow, pc/h
t_c = critical headway, seconds and
t_f = follow-up headway, seconds

Using the Nevada mean critical headways and follow-up headways obtained in Table 15 and Table 16 the capacity equations for Nevada roundabouts computed are presented as:

For single-lane entry conflicted by one circulating lane

$$C_{pce} = 1,230e^{(-0.67\times10^{-3})v_{c,pce}} \tag{19}$$

For double-lanes entries conflicted by one circulating lane

$$C_{pce} = 1,230e^{(-0.67\times10^{-3})v_{c,pce}} \tag{20}$$

For single-lane entry conflicted by two circulating lanes

$$C_{pce} = 1,231e^{(-0.95\times10^{-3})v_{c,pce}} \tag{21}$$

For double-lane entries conflicted by two circulating lanes

$$C_{e,L,pce} = 1,231e^{(-0.95\times10^{-3})v_{c,pce}} \tag{22}$$

$$C_{e,R,pce} = 1,221e^{(-0.92\times10^{-3})v_{c,pce}} \tag{23}$$

Using the Equations 19-23 for estimating the capacities for Nevada roundabouts will result in higher capacities than using the HCM, 2010 equations shown in Equations 11-15. It is necessary therefore to note these differences during roundabout design.

3.6 Comparison of Roundabout Analysis Software

As part of the project objectives, an investigation was conducted to compare some available roundabout analyses software using the operational indicator "delay" from field data as the benchmark. Two software, the "Highway Capacity Software" (HCS) which was developed along with the HCM 2010 and "SIDRA Solutions" a software developed by Akcelik and Associates, Australia were evaluated. Data extracted from the two single-lane roundabouts located in Carson City and Fernley were used for the software evaluation. Summary of 15-min field vehicle traffic volume counts reported as part of the output for the headways computation process were applied. The 15-minute volume counts from all the legs were entered into the two software systems along with the roundabout characteristics. Using the critical headway and follow-up headways obtained, the delays were computed from the respective software and compared with the field delays.

Figure 32 and Figure 33 are comparison plots of the delay obtained from the HCS, SIDRA and the field data. From the plots, SIDRA delay values appear closely matched to the field delay. The results suggest that SIDRA solutions software better predicts Nevada delay.

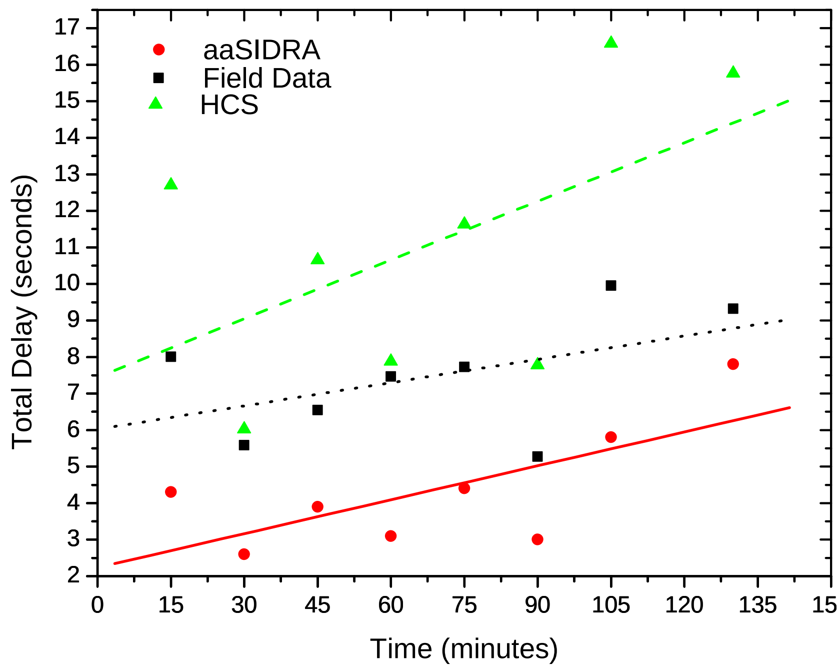


Figure 32: Comparison of Delays from HCS, SIDRA and the Field (Fairview Ave and Fifth St)

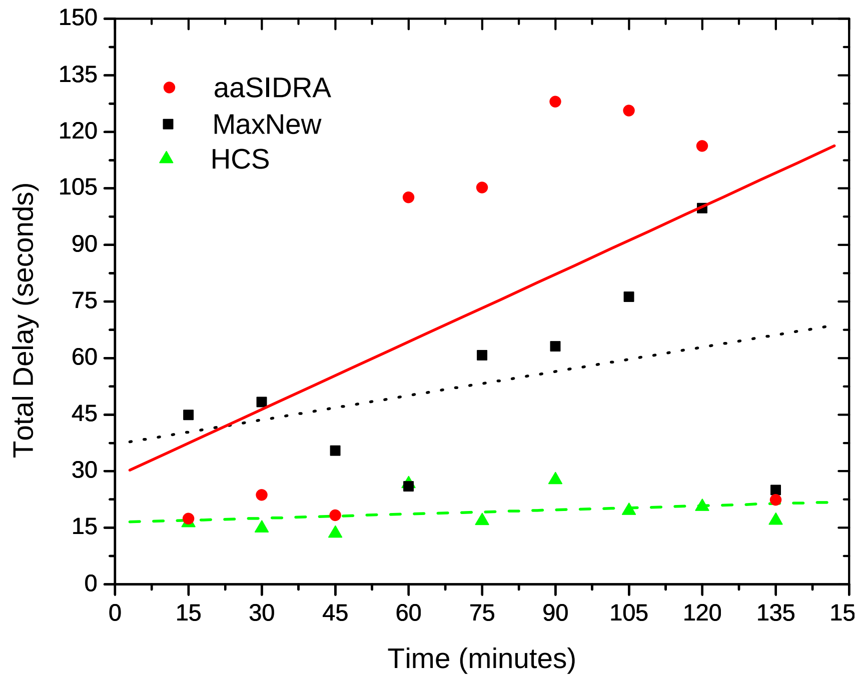


Figure 33: Comparison of Delays from HCS, SIDRA and the Field (State Route 343 and E Main St)

Figure 34 and Figure 35 show the delay values from the HCS and SIDRA plotted against field values. From the graphs, it can be seen that the results obtained from SIDRA appeared a better fit with the field data compared to HCS. It can be concluded that SIDRA is more suited for the prediction of delays at Nevada roundabouts.

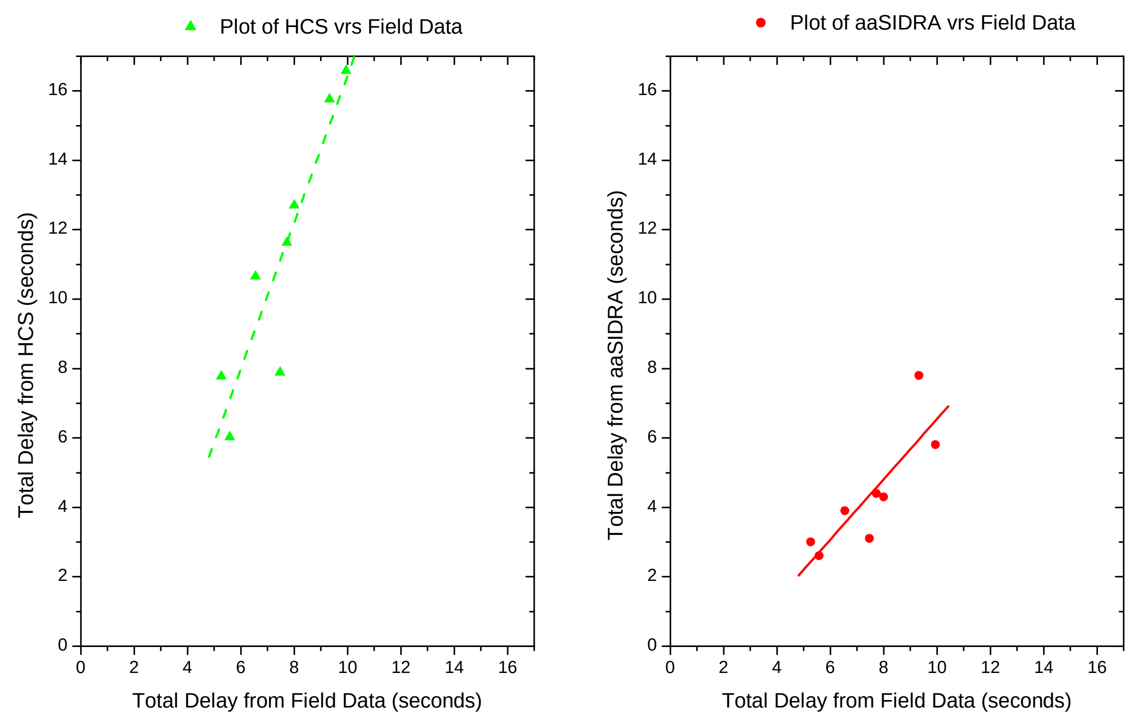


Figure 34: Comparison Plots of HCS and SIDRA vs. Field Data (Fairview Ave and Fifth St)

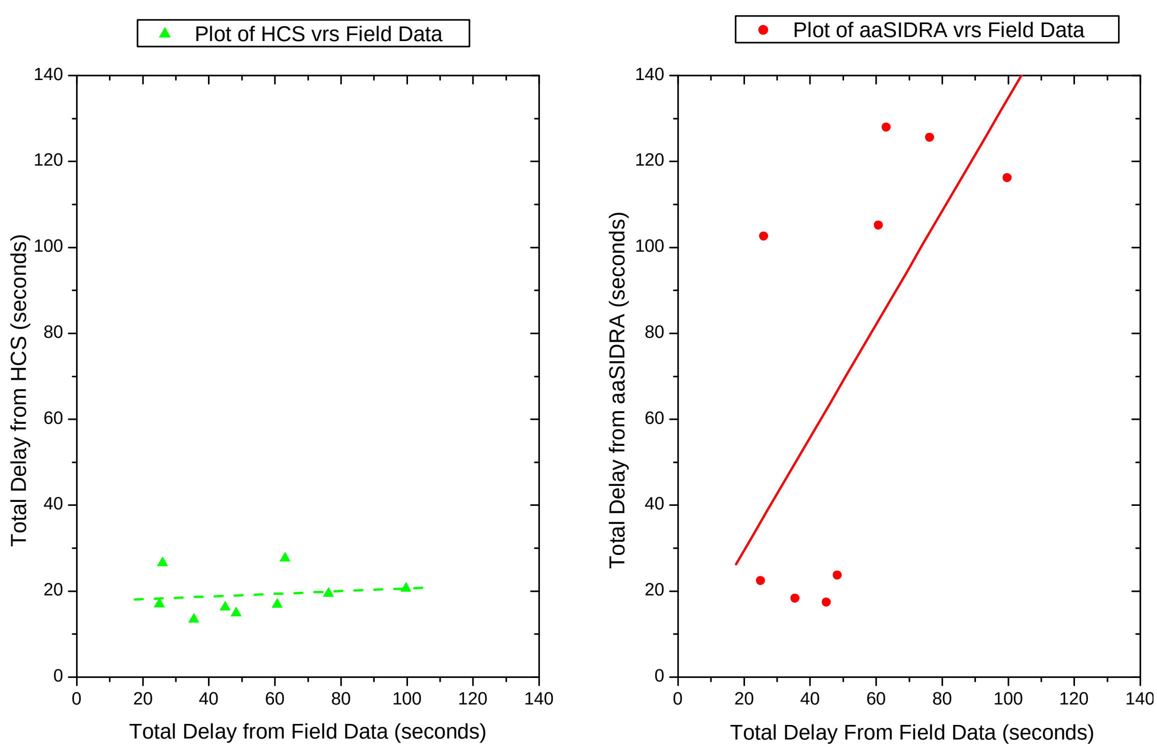


Figure 35: Comparison Plots of HCS and SIDRA vs. Field Data (State Route 343 and E Main St)

3.7 Summary of Findings

A summary of the main findings and the conclusion from the operational data analyses are presented below.

1. For single entry lane conflicted by single circulatory lane, the critical headway ranged between 3.3 and 4.4 seconds and the follow-up headway range between 2.7 and 3.3 seconds.

2. For the double-lane roundabout, the right-lane critical headways ranged between 4.3 and 5.5 seconds with a mean of 4.8 seconds, and the left-lane critical headways ranged between 4.1 and 5.9 seconds with a mean of 4.9 seconds. The right-lane follow-up headway ranged between 2.6 and 3.2 seconds with an average of 2.9 seconds and the left-lane follow-up headway ranged between 2.3 and 3.5 seconds with an average of 2.9 seconds.

3. Using the mean critical and follow-up headways for Nevada, the capacity models given in the HCM, 2012 have been calibrated for Nevada drivers and presented as:

For single-lane entry conflicted by one circulating lane

$$C_{pce} = 1{,}230e^{(-0.67\times10^{-3})v_{c,pce}}$$

For double-lanes entries conflicted by one circulating lane

$$C_{pce} = 1{,}230e^{(-0.67\times10^{-3})v_{c,pce}}$$

For single-lane entry conflicted by two circulating lanes

$$C_{pce} = 1{,}231e^{(-0.95\times10^{-3})v_{c,pce}}$$

For double-lane entries conflicted by two circulating lanes

$$C_{e,L,pce} = 1{,}231e^{(-0.95\times10^{-3})v_{c,pce}}$$

$$C_{e,R,pce} = 1{,}221e^{(-0.92\times10^{-3})v_{c,pce}}$$

4. SIDRA Solutions software appears to be more suited for analyses of Nevada roundabouts.

4 Roundabout Selection Guideline

4.1 Introduction

To select the best intersection control type, transportation professionals consider several factors including ROW, capacity, LOS, cost and safety of all control types before making a final decision. Intersection controls such as traffic signals, AWSC and TWSC have well established guidelines for determining when it is most appropriate to use them. Because roundabouts are relatively new in the U.S. and new state and local guidelines are still emerging, there is a substantial difference among jurisdictions when to select a roundabout. This research produced a new guideline to address situations in Nevada. The guideline will assist Nevada transportation professionals to determine when a roundabout should be used. The guideline consists of general site factors and operational factors.

4.2 General Site Factors

The literature review indicates that roundabouts can replace most existing AWSC and signalized intersections to achieve better LOS and safer operations. The life cycle cost of roundabouts varies widely from place to place but is generally lower than or comparable to signals. One desirable condition for roundabout construction is a relatively flat topography. Other locations for preliminary considerations of roundabouts include:

1. Intersections with a history of safety problems
2. Intersections with balanced approach traffic volumes
3. Intersections that must accommodate a high number of left turns and/or U-turns.
4. Intersections with high traffic volume at peak periods but relatively low volumes during non-peak periods
5. Intersections with more than four legs or unusual geometry such as “Y” or “T” configurations
6. At an entry point to a campus, neighborhood, or commercial development
7. Roads with a history of excessive speeds
8. Location with constrained queue storage
9. Existing two-way stop-controlled intersections with large side-street delays
10. Locations with a need to provide a transition between land use environments
11. Freeway interchange ramp terminals

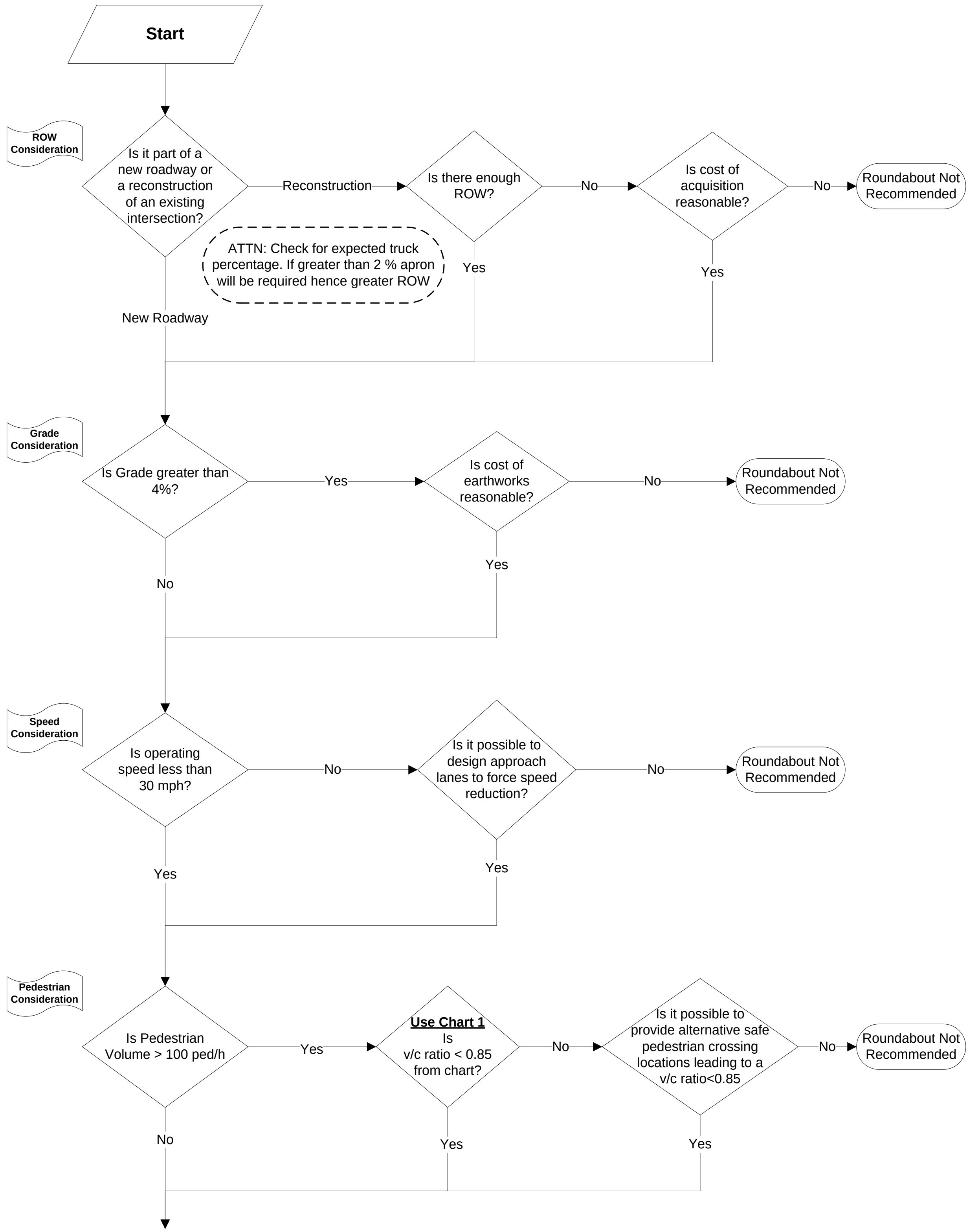
4.3 Operational Factors

Roundabout design requires several critical steps which can be time consuming and costly as other controls, therefore site feasibility needs thorough investigation before detailed designs are embarked upon. This section discusses further in-depth preliminary design procedures which precede final detailed design and requires basic information that usually exists or can be obtained with minimal effort. Data required are (but not limited to): ROW information, crash data, traffic volumes (peak hour and AADT for the design year), percentage of trucks, speed limits, pedestrian and bicyclist volumes (peak hour and AADT) and budget limit.

Using information from the NCHRP report 672 (9) and other state DOT's, a flowchart together with reference charts were developed for preliminary roundabout feasibility consideration. Figure 36 shows the flow chart and "Chart 1(A & B)" and "Chart 2 (A, B & C)" show the reference charts. The flow chart, divides the critical factors affecting the basic operations into eight consideration stages: 1) ROW, 2) Grade, 3) Speed, 4) Pedestrian, 5) Roadway, 6) Traffic Volume, 7) Safety and 8) Cost. Chart 1 (A & B) and Chart 2 (A & B) were developed based on the combined traffic volumes from the minor and major intersecting streets and help in decision-making at the pedestrian and volumes stages respectively.

Note that for roundabouts it is recommended to keep the volume-to-capacity (v/c) ratio less than or equal to 0.85. At the pedestrian and traffic volume consideration stages, the v/c is evaluated using the vehicular volumes converted to passenger car units (PCU). Chart 1A and 1B give transportation engineers a quick indication of the pedestrian volume effect on the capacity of single-lane and double lane roundabouts respectively. At the traffic volume consideration stage, Chart 2A and 2B together with chart 2C are used to suggest which control type is the best. Charts 2A and 2B combine the major and minor street peak hour volumes to determine the best control based on the LOS and v/c ratio < 0.85 for single-lane and double-lane roundabouts respectively. Chart 2C adopted from the NCHRP report 672 (9) uses the AADT to guide designers select the required number of lanes. The detail of the development process for Charts 1 (A & B) and 2 (A & B) appear in Appendix A.

Consulting Figure 36, users begin with the "START" stage and proceed through the flowchart answering the various questions. Based on the intersection and traffic information, the answers to the flowchart questions lead users to decide at each stage whether to continue considering a roundabout. If at any stage roundabout is not recommended, the process is terminated and an alternative control should be investigated. If roundabout emerges as the most appropriate control at the end of the process, only then should detailed design be commenced.



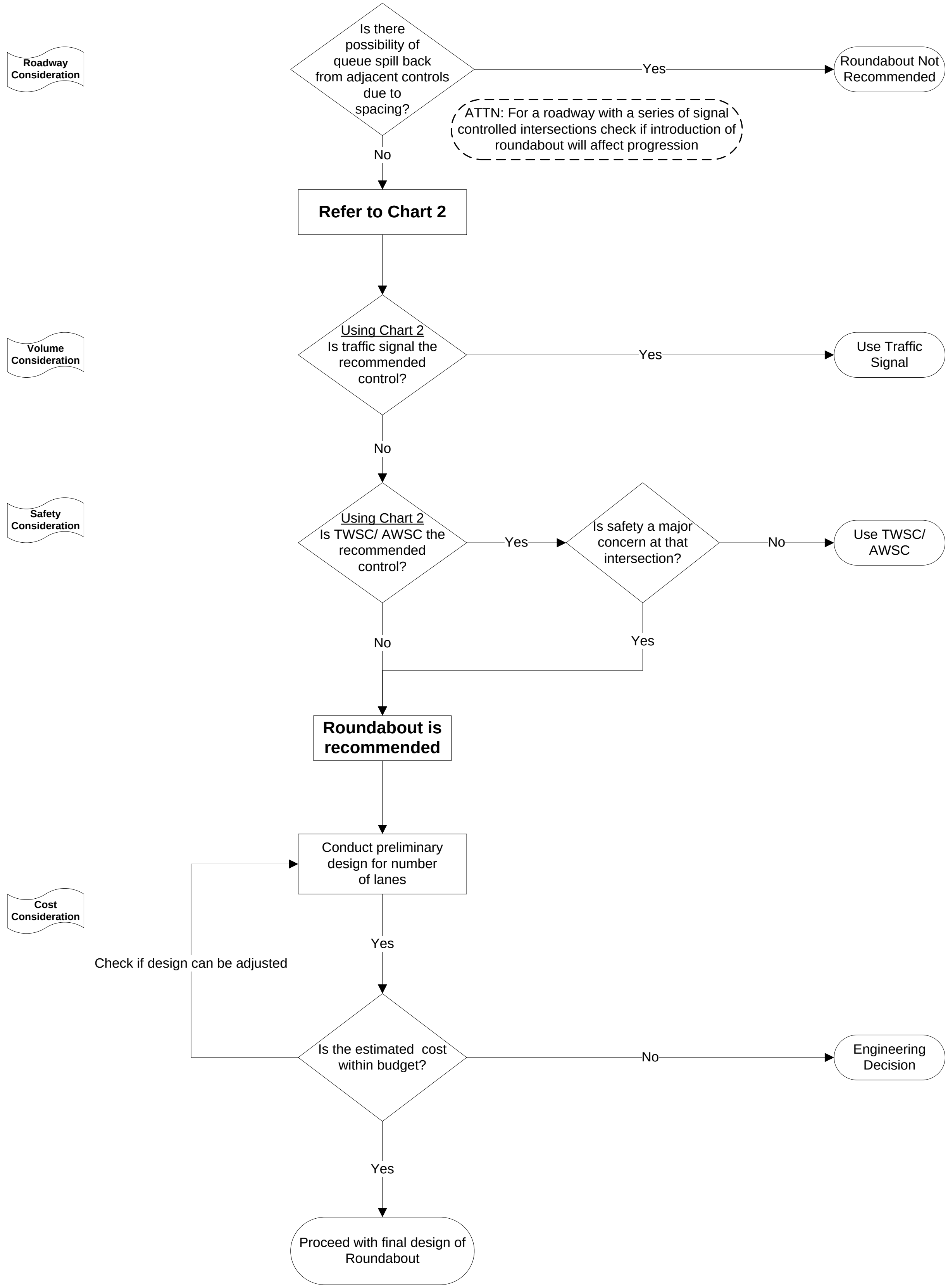


Figure 36: Flow Chart for Preliminary Selection of Roundabout as an Intersection Control

CHART 1A Pedestrian Effect on Entry Capacity (Single-Lane Roundabouts)

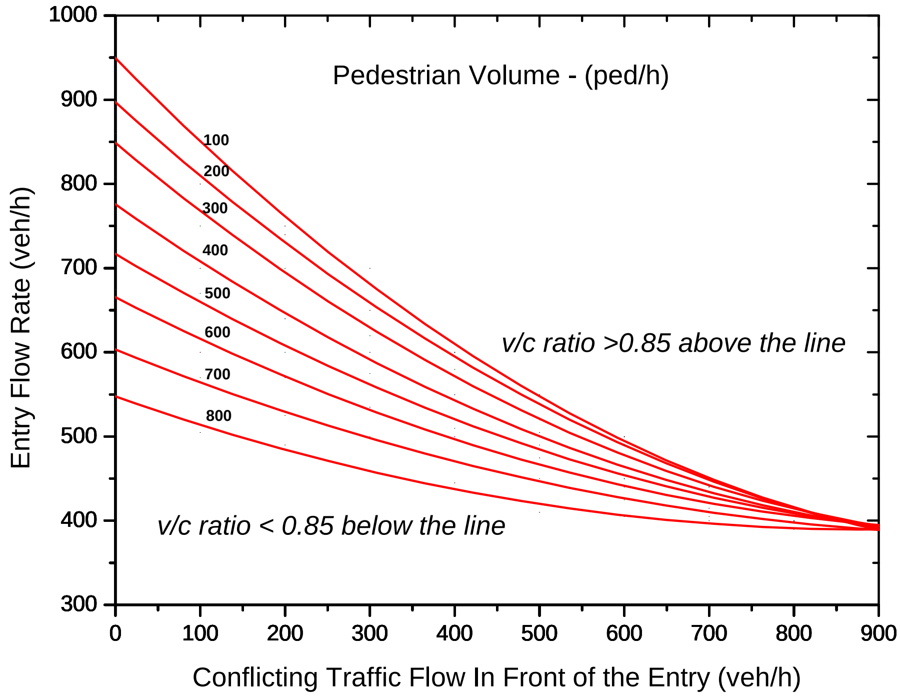


CHART 1B Pedestrian Effect on Entry Capacity (Double-Lane Roundabouts)

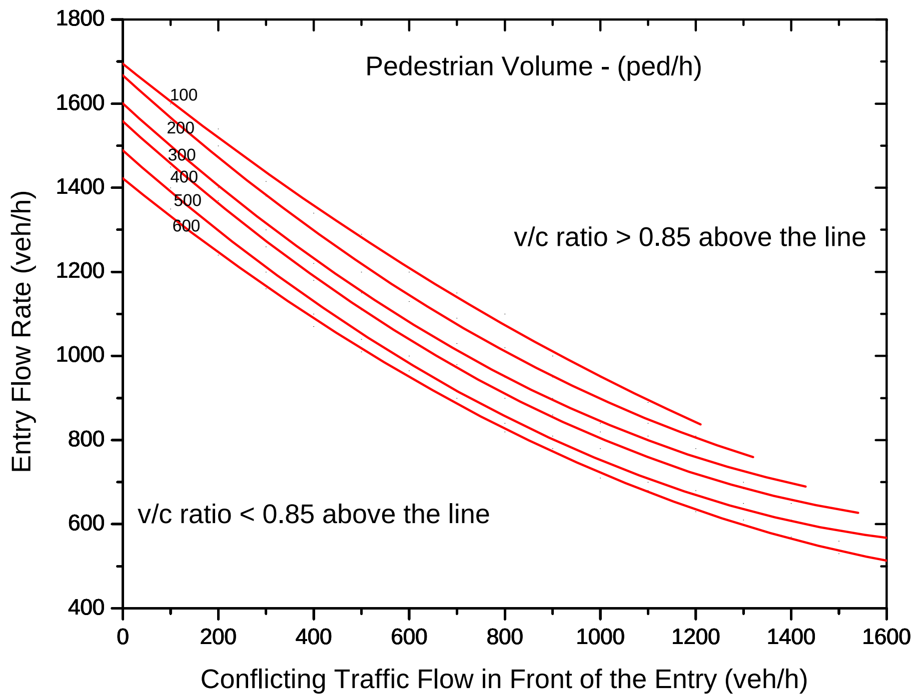
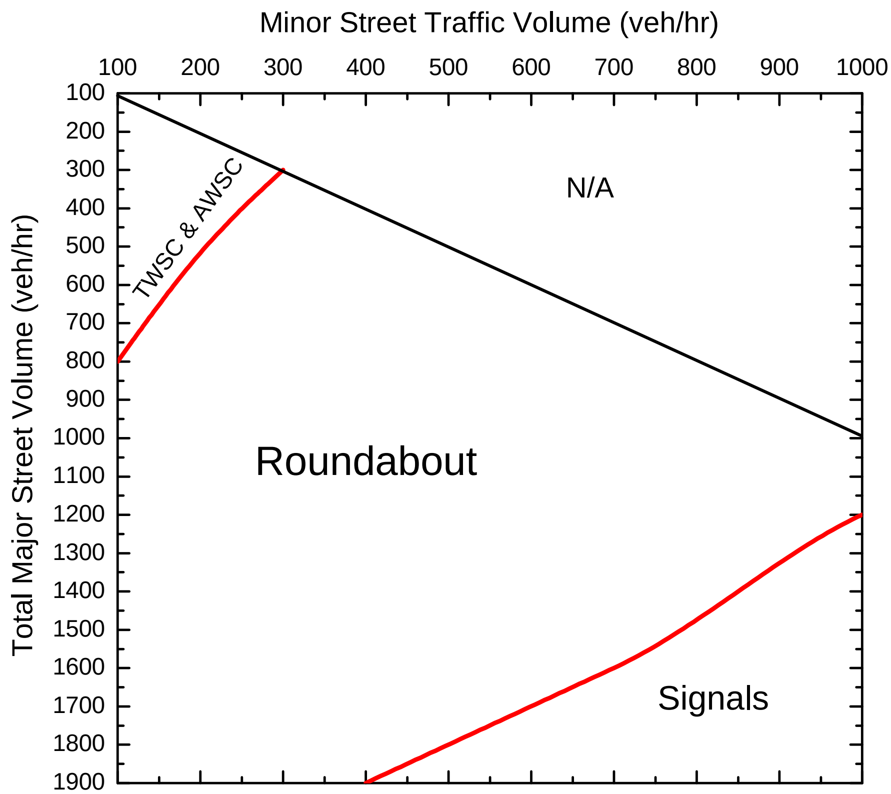
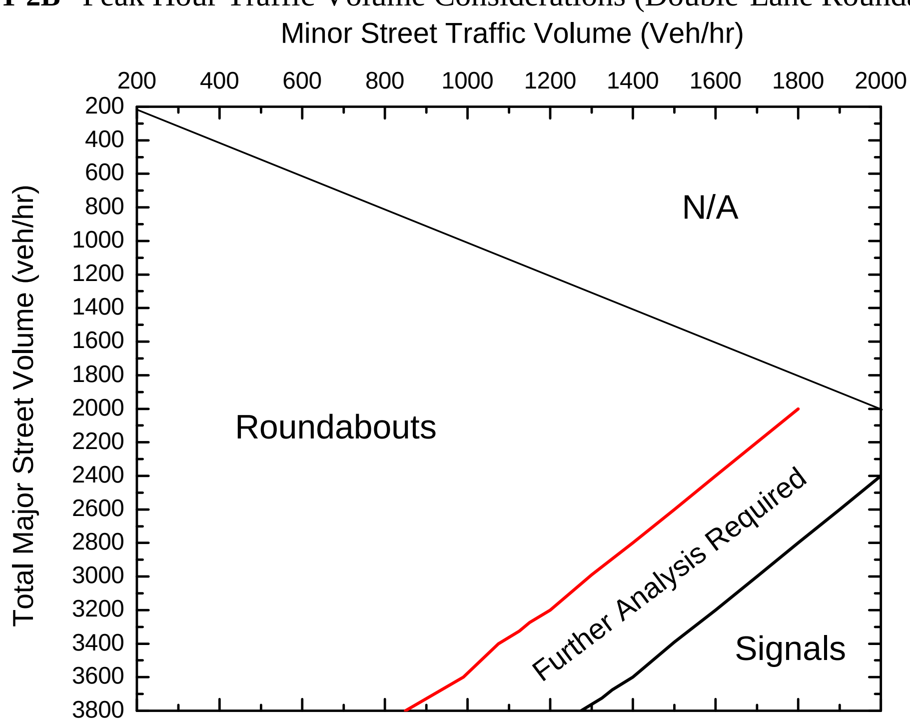


CHART 2A – Peak Hour Traffic Volume Considerations (Single-Lane Roundabouts)



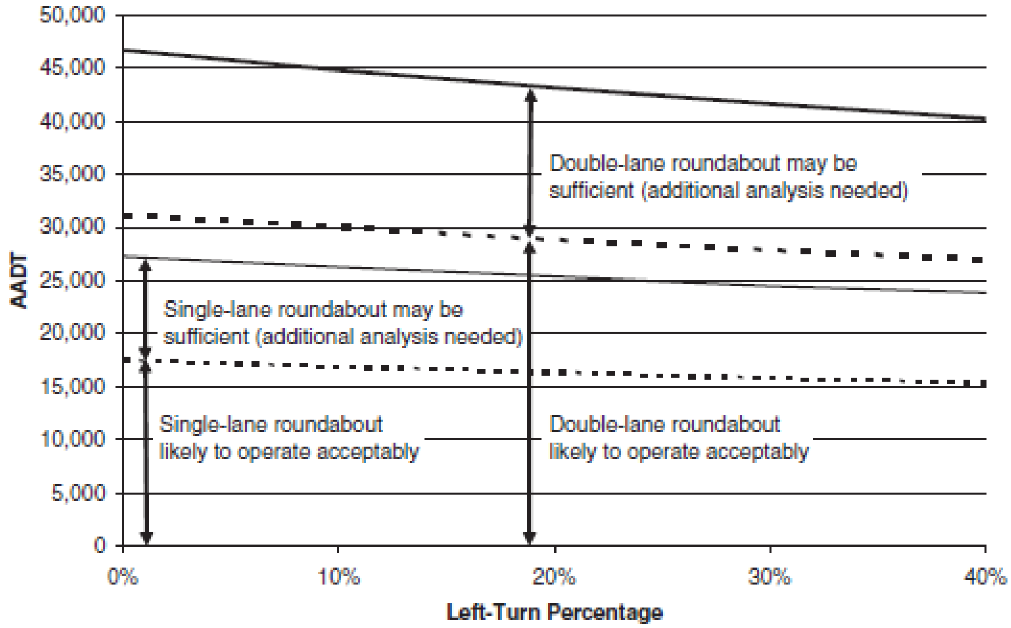
N/A - the chart is not applicable within that region because there is reversal of roles

CHART 2B –Peak Hour Traffic Volume Considerations (Double-Lane Roundabouts)



N/A - the chart is not applicable within that region because there is reversal of roles

CHART 2C – Daily Intersection Volume (AADT)



Source: NCHRP Report 672

5 Findings, Recommendations and Implementation plan

This report contains a comprehensive literature review, data collection, extraction and analyses, and a roundabout selection guideline procedure. The findings and recommendations reached are summarized in this section. Additionally, implementation activities are included for NDOT to consider. These findings, recommendations and implementation plan are based on and are limited to the data collected and analyzed for this project.

5.1 Findings

1. The following are the mean values obtained for Nevada critical headways and follow-up headways and should be used for design in place of the national averages reported in the NCHRP 3-65 Project (NCHRP 572 report).
    a. For single-lane roundabouts: critical headway = 3.9 seconds, follow-up headway = 2.9 seconds
    b. For double-lane roundabouts, left lane: critical headway = 4.9 seconds, follow-up headway = 2.9 seconds
    c. For double-lane roundabouts, right lane: critical headway = 4.8 seconds, follow-up headway = 2.9 seconds
2. Nevada drivers exhibit a follow-up headway that is significantly less than the follow-up headways exhibited by drivers from other states as presented in the NCHRP 3-65 Project
3. Nevada drivers have a critical headway that is not significantly different than critical headways obtained from other states by the NCHRP 3-65 Project.
4. A decision flowchart with supporting supplementary charts has been developed to help determine when roundabouts control should be considered in the Nevada.

5.2 Recommendations

1. It is recommended that NDOT tests, refines, and adopts the decision flowchart developed in this research for selecting roundabouts as an intersection control type.
2. The following calibrated capacity models are recommended for use in Nevada:
    a. For single-lane roundabouts:

$$C_{pce} = 1{,}230e^{(-0.67\times10^{-3})v_{c,pce}}$$

b. For two-lane roundabout, (left lane):

$$C_{e,L,pce} = 1{,}231e^{(-0.95\times10^{-3})v_{c,pce}}$$

c. For two-lane roundabout, (right lane):

$$C_{e,R,pce} = 1{,}221e^{(-0.92\times10^{-3})v_{c,pce}}$$

3. "SIDRA Solutions" is recommended as the preferred roundabout analysis software for use in Nevada

5.3 Implementation Plan

The implementation plan for this research project consists of two steps:

1. Distribute the recommended guidelines in Appendix B internally to selected traffic engineers to test the guidelines on a few intersections. Examples on the use of the recommended guidelines are presented in Appendix B.
2. There is the need to schedule workshops for NDOT, local/regional agencies and consultant transportation engineering personnel.

APPENDIX A

Development of Charts 1 (A &B) and Charts 2 (A &B)

A1 – Development of “Pedestrian Effect on Entry Capacity” Charts

Charts 1 (A & B) are for estimating capacity loss on an entry leg due to pedestrian crossing effects and also indicate the volume-to-capacity (v/c) ratio. These were derived using equations developed by the Brilon, Stuwe and Drews (65) based on the combined entry and circulatory traffic volumes. In this formula, the entry capacity “C” (obtained from a procedure which does not include pedestrian crossing) is reduced by a factor “M”, where M is the reduction effect of the pedestrian crossing rate. The new capacity “C_{ped}” is given as:

$$C_{ped} = C \times M$$

For a single-lane entry;

$$M = \frac{1119.5 - 0.715 \times Q_c - 0.644Q_{ped} - 0.00073Q_cQ_{ped}}{1069 - 0.65Q_c}$$

For a double-lane entry;

$$M = \frac{1260.6 - 0.381Q_{ped} - 0.329Q_c}{1380 - 0.50Q_c}$$

Where

Q_c = circulating flow in front of the entry (pcu/h)

Q_{ped} = pedestrian flow crossing the leg (ped/h)

It must be noted that the equations for “M” are also used in the HCM 2010 (53).

The “M” equations are most appropriate for Q_{ped} range between 100-600 ped/h and may require further research for higher volumes. For pedestrian volume lower than 100 ped/h the reduction factor “M” is about 0.99 and has insignificant effect on the entry capacity. Using the equations for “M” above, the values shown in the Table 18 for single-lane entries and Table 19 for double-lane entries were obtained for the respective circulatory flow rates. The table may be interpolated and the “M” values multiplied by the capacity (without pedestrian effect) to compute the required capacity C_{ped}.

Table 18 M Values for Single – Lane Entry

Pedestrian crossing the leg (ped/h)	Circulating Flow in Front of the Entry (pcu/h)									
	0	**100**	**200**	**300**	**400**	**500**	**600**	**700**	**800**	**870**
100	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
200	0.93	0.93	0.93	0.94	0.94	0.95	0.96	0.96	0.98	0.98
300	0.87	0.87	0.88	0.89	0.90	0.91	0.93	0.94	0.96	0.98
400	0.81	0.82	0.83	0.84	0.86	0.87	0.90	0.92	0.95	0.98
500	0.75	0.76	0.77	0.79	0.81	0.84	0.87	0.90	0.94	0.98
600	0.69	0.70	0.72	0.74	0.77	0.80	0.83	0.88	0.93	0.98

Table 19 M Values for Multi – Lane Entry

Pedestrian crossing the leg (ped/h)	Circulating Flow in Front of the Entry (pcu/h)									
	0	**200**	**400**	**600**	**800**	**1000**	**1200**	**1400**	**1600**	**1800**
100	0.89	0.90	0.92	0.95	0.98	1.02	1.06	1.12	1.20	1.31
200	0.86	0.87	0.89	0.91	0.94	0.97	1.01	1.06	1.13	1.23
300	0.83	0.84	0.86	0.88	0.90	0.93	0.96	1.01	1.07	1.15
400	0.80	0.81	0.83	0.84	0.86	0.89	0.91	0.95	1.00	1.08
500	0.78	0.78	0.80	0.81	0.82	0.84	0.87	0.90	0.94	1.00
600	0.75	0.75	0.76	0.77	0.78	0.80	0.82	0.84	0.87	0.92

The HCM, 2010 capacity equations for entry lanes are given below:

For single lane

$$C_{pce} = 1{,}130e^{(-1.0\times10^{-3})v_{c,pce}}$$

For double lane

$$C_{e,R,pce} = 1{,}130e^{(-0.7\times10^{-3})v_{c,pce}}$$

$$C_{e,L,pce} = 1{,}130e^{(-0.75\times10^{-3})v_{c,pce}}$$

Where
C_{pce} = lane capacity, adjusted for heavy vehicles, pc/h;
$C_{e,R,pce}$ = capacity of right entry lane, adjusted for heavy vehicles, pc/h;
$C_{e,L,pce}$ = capacity of left entry lane, adjusted for heavy vehicles, pc/h; and
$v_{c,pce}$ = conflicting flow pc/h.

The capacity taking the pedestrian effect into consideration is given below (for single-lane roundabouts):

$$C_{/ped} = M \times 1130e^{(-1.0\times10^{-3})V_{c,pce}}$$

The capacities computed from the HCM capacity equation were multiplied by the reduction factors obtained from the Table 18 and Table 19. New volume-to-capacity (v/c) ratios were computed by divided the "C_{ped}" by the "C" obtained. Table 20 shows the pedestrian effect on the roundabout entry lane capacity and Table 21 shows the v/c ratio for different volume combinations for 100 pedestrian. Charts 1 (A and B) were developed using different pedestrian volumes and varying the entry and conflicting traffic volumes in front of the entry lane(s). These lines were drawn to trace a path for the v/c ratio equals 0.85. Above the respective pedestrian volume lines, the v/c is greater than 0.85. If the volume combinations give results very close to the line, further analyses are recommended before making a final decision. The charts were developed based on passenger car units (PCU)

Table 20 Pedestrian Volume Effects on Entry-Lane Capacity

Pedestrian Flow crossing the leg (ped/h)	Circulating Flow in Front of the Entry (pcu/h): 0	100	200	300	400	500	600	700	800	870
100	1115	1009	913	826	747	676	612	554	501	467
200	1047	951	864	785	714	651	593	541	495	466
300	979	893	815	745	682	625	574	529	490	465
400	911	835	766	704	649	599	555	517	484	464
500	843	777	717	663	616	573	537	505	479	464
600	775	718	668	623	583	548	518	493	473	463

Table 21 Volume-to-Capacity Ratio at Roundabout Entry: 100 Pedestrians

Entry Flow rate (pcu/h)	Circulating Flow in Front of the Entry (pcu/h): 0	100	200	300	400	500	600	700	800	900
100	0.09	0.10	0.11	0.12	0.13	0.15	0.16	0.18	0.20	0.21
200	0.18	0.20	0.22	0.24	0.27	0.30	0.33	0.36	0.40	0.43
300	0.27	0.30	0.33	0.36	0.40	0.44	0.49	0.54	0.60	0.64
400	0.36	0.40	0.44	0.48	0.54	0.59	0.65	0.72	0.80	0.86
500	0.45	0.50	0.55	0.61	0.67	0.74	0.82	0.90	1.00	1.07
600	0.54	0.59	0.66	0.73	0.80	0.89	0.98	1.08	1.20	1.29
700	0.63	0.69	0.77	0.85	0.94	1.04	1.14	1.26	1.40	1.50
800	0.72	0.79	0.88	0.97	1.07	1.18	1.31	1.45	1.60	1.71
900	0.81	0.89	0.99	1.09	1.20	1.33	1.47	1.63	1.80	1.93
1000	0.90	0.99	1.10	1.21	1.34	1.48	1.63	1.81	2.00	2.14

CHART 1A Pedestrian Effect on Entry Capacity (Single-Lane Roundabouts)

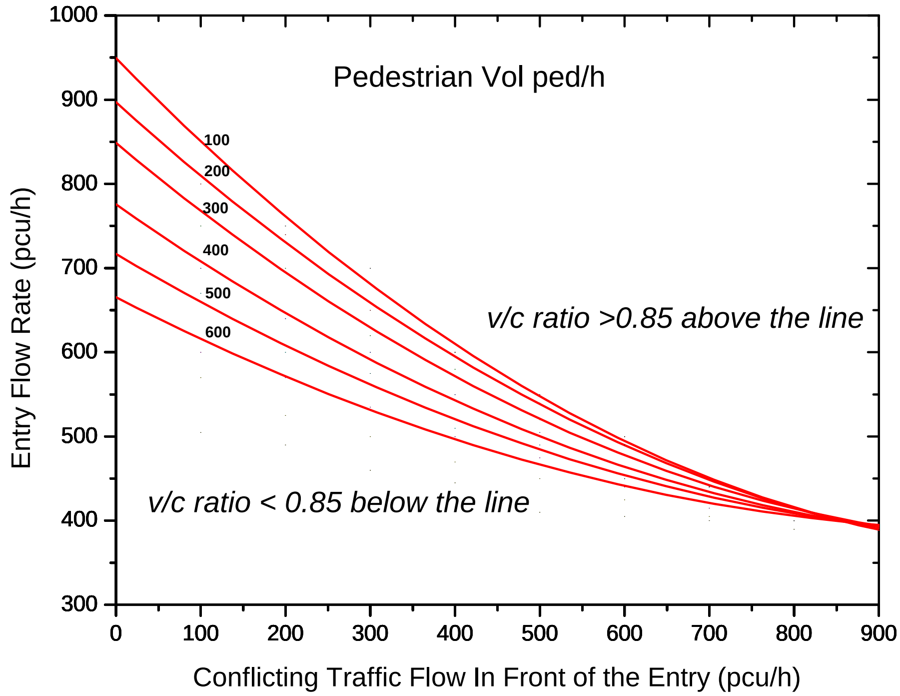
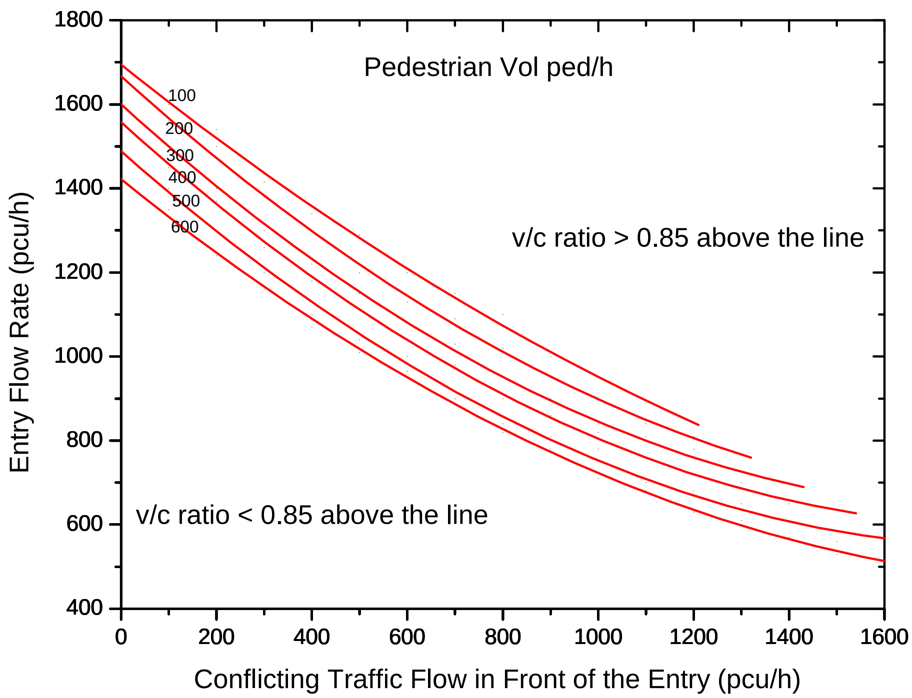


CHART 1B Pedestrian Effect on Entry Capacity (Double-Lane Roundabouts)



A2 – Development of "Intersection Control Selection" Charts

"Charts 2 A and 2 B" were developed by comparing the performance of TWSC, AWSC, roundabouts and signals controls. Generic intersections were used to compare the performance analysis models in chapters 18 to 21 of the HCM 2010; adopted from Marek et al (66) and modified to include parameters for roundabouts. The intersection geometries were based on field observations. Figure 37 shows the characteristics of the single-lane approach intersections.

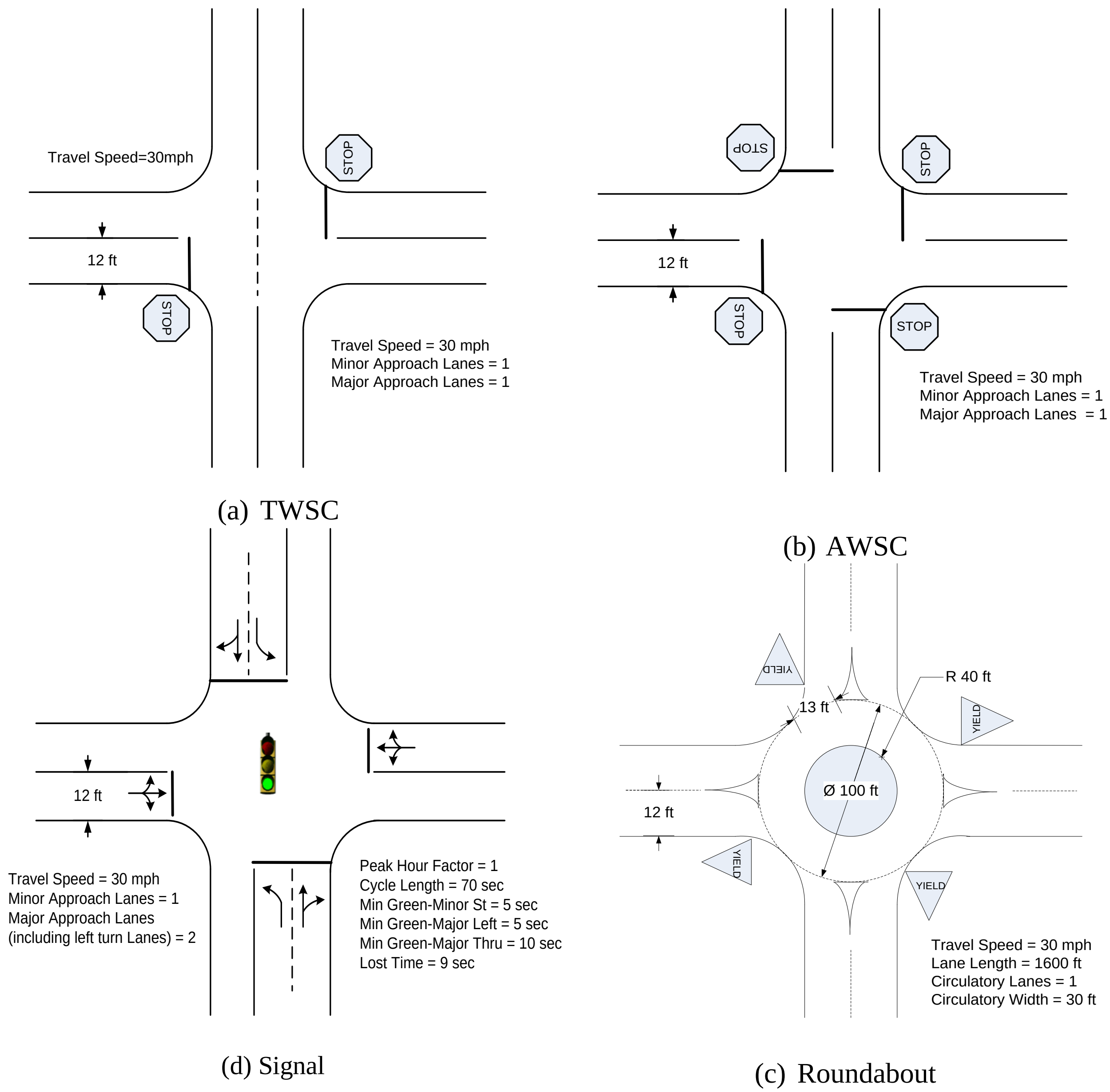


Figure 37: Characteristics of Generic Intersections (Signalized and Roundabout)

Using *Figures 4C-1* to *Figures 4C-4* from the Manual of Uniform Traffic Control Devices (MUTCD) 2009 (61), the major street approach volumes ranged from 100 to 1900 vehicles per hour (veh/hr) and the minor street approach volumes ranged from 100 to 1000 veh/hr in increments of 100 veh/hr. For the double-lane roundabout, the major street approach volume ranged from 200 to 3800 veh/hr and the minor street approach volume ranged from 200 to 2000 veh/hr in increments of 200 veh/hr. There were 164 total combinations for both roundabout types. The volume splits for the turning movements on the approaches were also adopted from Marek et al (66) and shown in Table 22.

Table 22 Intersection Volume Split

Direction	Total Split	Left Split	Thru Split	Right Split
Major Street	%			
Major 1	50	10	75	15
Major 2	50	15	70	15
Minor Street	%			
Minor 1 (subject)	70	20	40	40
Minor 2	30	25	50	25

To obtain the LOS and average intersection delay, SIDRA intersections software version 5.0 was used to analyze and compare all four control types. SYNCHRO Traffic software version 6 was used to analyze the TWSC, AWSC and signalized intersections and the results used to verify results from SIDRA. (The SYNCHRO HCM report values were used in this analysis). The combined major and minor street approach volumes were used to for analyses. The volumes were varied on both the major and minor streets to give different total volume combinations before analyzing each control type. Comparisons were based on level of service, delay, and queue length. For each volume combination, the best intersection control was chosen based on all three performance criteria. Table 23 is an example of the calculation and analysis process. This table shows the performance output for the different control types when the volume combinations are altered for all four control types (major street volume is kept at 700 veh/hr and minor street volume ranged from 200-500 veh/hr). The control(s) with the best performance measure is selected as the optimal control type.

Table 23 Sample Calculations for Selecting Optimal Control Type

TYPE 1: Optimal Intersection Control Based on LOS

Major St Vol (vph)	Minor St Vol (vph)	TWSC LOS	AWSC LOS	Roundabout LOS	Signal LOS	Optimal Control Type
700	200	C	B	A	A	R, S
700	300	D	B	A	B	R
700	400	E	C	A	B	R
700	500	E	C	A	B	R

TYPE 2: Optimal Intersection Control Based on Average Control Delay

Major St Vol (vph)	Minor St Vol (vph)	TWSC LOS	AWSC LOS	Roundabout LOS	Signal LOS	Optimal Control Type
700	200	5.2	11.8	5.5	9.1	T
700	300	9	14.2	5.9	12.1	R
700	400	13.4	15.2	6.3	14.1	R
700	500	18	18.3	6.7	16.2	R

TYPE 1: Optimal Intersection Control Based on Average Queue Length

Major St Vol (vph)	Minor St Vol (vph)	TWSC LOS	AWSC LOS	Roundabout LOS	Signal LOS	Optimal Control Type
700	200	1.3	2.95	1.38	2.28	T
700	300	2.5	3.94	1.64	3.36	R
700	400	4.01	4.64	1.93	4.31	R
700	500	6	6.1	2.23	5.4	R

Where

R = Roundabouts
S = Signal
T = TWSC
A = AWSC

From the analysis, table were developed indicating the best control type based on the selected performance criteria chosen using combined major and minor street volumes. Table **24** is an example of the optimal intersection control table based on level of service.

Charts 2A and 2B were developed for single-lane and double-lane roundabouts respectively based on similar results obtained for all the three performance criteria. The charts are a quick reference for determining if roundabouts are applicable based on using volume combinations only.

Table 24 Optimal Intersection Control Based on Level of Service

Comparing TWSC, AWSC, RA and Signals

Total Major Street (veh/hr)	Highest Minor Street Volume (veh/hr): 70	140	210	280	350	420	490	560	630	700
100	TARS									
200	TARS	ARS								
300	ARS	ARS	AR							
400	ARS	ARS	AR	R						
500	ARS	ARS	R	R	R					
600	ARS	RS	R	R	R	R				
700	RS	RS	R	R	R	R	R			
800	RS	RS	R	R	R	R	R	R		
900	RS	RS	R	R	R	R	R	R	R	
1000	RS	R	R	R	R	R	R	R	R	R
1100	RS	R	R	R	R	R	R	R	R	R
1200	RS	R	R	R	R	R	R	R	RS	R
1300	RS	R	R	R	R	R	R	R	RS	RS
1400	RS	R	R	R	R	R	R	RS	R	RS
1500	RS	R	R	R	R	R	R	RS	RS	S
1600	RS	R	R	RS	R	R	RS	RS	S	RS
1700	RS	R	R	RS	R	RS	S	RS	S	S
1800	RS	R	R	RS	RS	RS	RS	S	S	S

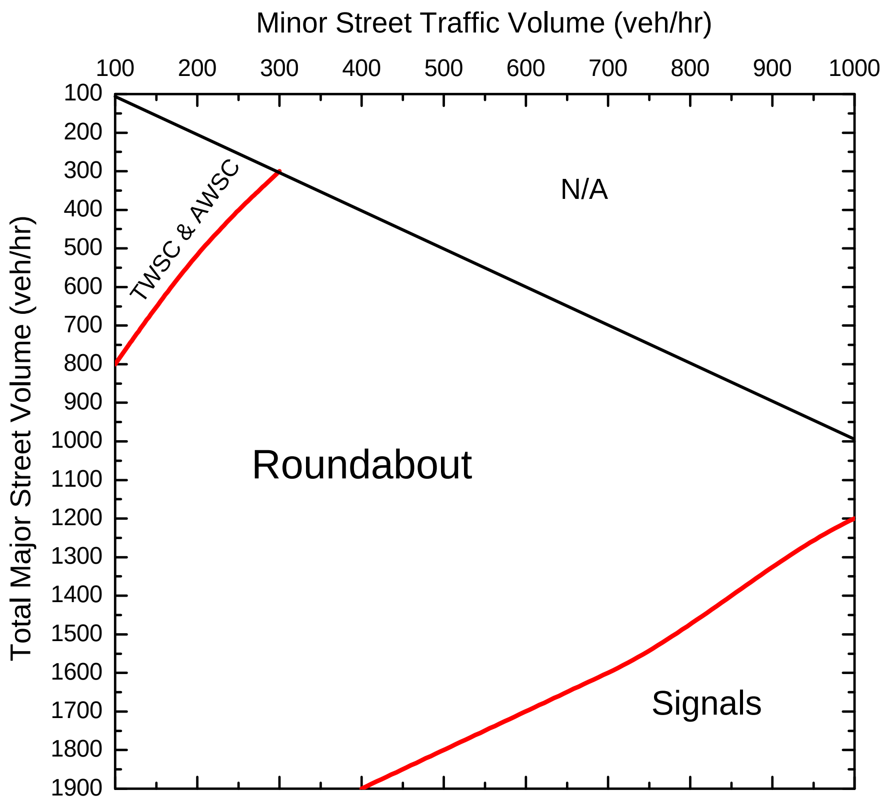
(a)

Comparing TWSC, AWSC and Signals

Total Major Street (veh/hr)	Highest Minor Street Volume (veh/hr): 70	140	210	280	350	420	490	560	630	700
100	TAS									
200	AS	AS								
300	AS	AS	A							
400	AS	AS	A	AS						
500	AS	AS	AS	AS	AS					
600	AS	AS	AS	AS	S	S				
700	S	S	AS	S	S	S	S			
800	S	S	S	S	S	S	S	S		
900	S	S	S	S	S	S	S	S	S	
1000	S	S	S	S	S	S	S	S	S	S
1100	S	S	S	S	S	S	S	S	S	S
1200	S	S	S	S	S	S	S	S	S	S
1300	S	S	S	S	S	S	S	S	S	S
1400	S	S	S	S	S	S	S	S	S	S
1500	S	S	S	S	S	S	S	S	S	S
1600	S	S	S	S	S	S	S	S	S	S
1700	S	S	S	S	S	S	S	S	S	S
1800	S	S	S	S	S	S	S	S	S	S

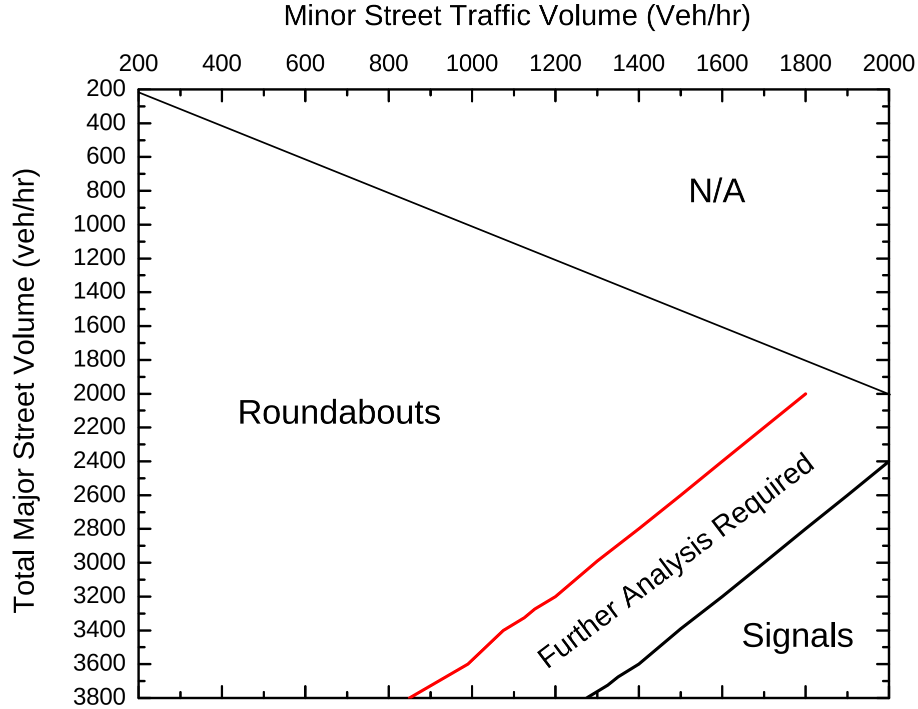
(b)

CHART 2A – Peak Hour Traffic Volume Considerations for Single-Lane Roundabouts



N/A - the chart is not applicable within that region because there is reversal of roles

CHART 2B – Peak Hour Traffic Volume Considerations for Double-Lane Roundabouts



N/A - the chart is not applicable within that region because there is reversal of roles

APPENDIX B

Supplementary Guidance for Flowchart Implementation

Introduction

This section showcases tests of the flowchart using information from intersections in Nevada. The objectives of the case studies were to verify the applicability of the flowchart and the supporting figures using real situations. NDOT may in addition to these examples set up teams of transportation professionals to also use the flow chart tool for selected sites that are potential roundabout locations. This will test the suitability or otherwise of the flowchart. Following a successful trial and pending any modification, the flowchart may then be adopted as a working tool for the State of Nevada. There are three cases studies using data from 2011.

Case 1

The Oddie Boulevard (Blvd) and Sutro Street (St) intersection near the Rodeo Event Center in Reno is controlled by a traffic signal. It is desired to test the suitability of using a roundabout as a control for the intersection because of the high number of pedestrian that visit the Event Center during events. The intersection has two lanes with a short left turn pocket on each approach. The 2011 AADT on Oddie Blvd and Sutro St are 14,000 vehicles and 13,000 vehicles respectively and the left turn volumes are 25 percent for Oddie and 12 percent for Sutro St. The peak period traffic volumes are 1,300 vehicles per hour (vph) on Oddie Blvd and 1,100 vph on Sutro St with about 2 percent truck volume for both streets.

A visual inspection of the site revealed enough ROW near the intersection. The speed limits are 35 mph on Oddie and 30 mph on Sutro St. Pedestrian volume is lower than 100 pedestrians per hour (pph) on normal days but increases to over 400 pph during events days for the Rodeo Event Center which is situated at one of the quadrants. NDOT crash data showed 2 pedestrian related crashes in the past 4 years. Also, during events, long delays and queues are experienced at the intersection which was needed to be addressed since it is projected that the traffic volume will grow with improvement in the economy.

To determine if a roundabout is suitable for the intersection, Figure 36 (flow chart) was used and Table 25 shows answers to the questions at each of the eight consideration stages. The flowchart reduces the time required for analysis whiles eliminating discrepancies by giving guidance to Engineers and other professionals. In conclusions, Engineers are better guided through the process of selecting an intersection control type with a major time savings.

Table 25 Typical Answers Derived Using the Roundabout Guideline (Case 1)

No	Consideration	Discussion	Roundabout Decision
1	Right of way	The location currently is able to contain two lanes and a left turn pocket. This implies there might be sufficient ROW. If more land is required, it is possible to obtain extra land since adjoining lands are occupied by gas stations, a 7/11 convenient store, a parking lot for the Rodeo event center and a residency. All should be easy to obtain.	Continue consideration
2	Grade	The land area is fairly flat with a gentle slope towards the north of Sutro St. Minimal earthworks required since visibility is good	Continue consideration
3	Speed	The speeds on the cross streets differ by 5 mph. the street ROW are wide enough to allow speed reduction measures	Continue consideration
4	Pedestrian	On normal days, pedestrian volume is easy to accommodate. On event days, pedestrian issues excludes single-lane roundabout since the v/c ratio is greater than 0.85 but double lane roundabouts suffice. See Charts 1 (A& B).	Continue consideration (double-lane roundabouts)
5	Roadway	The closest intersection is the traffic signal on the north of Sutro St and is over 1,800 ft away. No queue spill back is expected.	Continue consideration
6	Traffic volume	Considering the peak traffic volume combination and the AADT, a double-lane roundabout is appropriate. The combined AADT is 27,000 vehicles and the left turn percentage is 23 percent on Oddie Blvd. The peak hour traffic volume combination also recommends roundabout. See Charts 2 (A – C).	Continue consideration (double-lane roundabouts)
7	Safety	There is no current safety treatment at the intersection but double-lane roundabout provides enough refuge for pedestrians at the splitter island allowing for two-stage crossing which is safer for pedestrians	Continue consideration
8	Cost	From operation and safety point, a roundabout is appropriate but since there is no budget allocation, the final decision can be made after initial cost estimates are done. Considering the data, it is most likely the cost of roundabout might be average.	Roundabout is feasible

Case 2

There are growing concerns for the safety of pedestrians (mainly students and faculty) that cross the Evans Avenue (Ave) at the intersection of Evans Ave and Record Street (St) on the southern end of the University of Nevada, Reno. The intersection is currently controlled by an AWSC but roundabout is a proposed alternative for safety reasons. The 2011 AADT on Evans Ave and Record St were 2600 vehicles and 1100 vehicles and the peak period traffic count yielded 573 and 59 vph respectively. A pedestrian volume of 280 pph was recorded during the peak period. Speed limits on Evan Ave and Record St for that section are 25 mph and 15 mph respectively. About 150 feet on the west bound approach (Evans St) is a 70 degree curve and approximately 10 percent gradient over a distance of 120 ft. The general land use around the intersection is car parking lots. One quadrant however has a single story office building. Using the flow chart to verify the appropriateness of a roundabout at the location gave the responses shown in Table 26 for the eight consideration stages. The recommendation is that a roundabout is appropriate for the intersection if the cost of reducing the steep grade/curve on the west bound approach can be accommodated within the budget.

Table 26 Typical Answers Derived Using the Roundabout Guideline (Case 2)

No	Consideration	Discussion	Roundabout Decision
1	Right of way	The location has single lanes in each direction with short left turn pockets on three of the approaches. This makes room for the inscribed circle of the roundabout to be constructed with little additional land requirement if any. There might be enough ROW.	Continue consideration
2	Grade	The slope grade is very steep along with the curve on the eastern approach so will require major earthworks to correct the grade and improve visibility. (The budget will be a determining factor)	Proceed with roundabout only if budget can accommodate the earthworks needed to improve visibility and reduce gradient
3	Speed	The speeds on Evans Ave is 25 mph and on Record is 15 mph. Speed reduction is needed on Evan Ave to improve safety	Continue consideration
4	Pedestrian	The pedestrian volume effect on the v/c ratio is acceptable as the v/c ratio does not exceed 0.85 with minimal effect on capacity. Pedestrians will also be safer with lower speeds from the roundabouts. Chart 1A is used.	Continue consideration
5	Roadway	The closest intersections on Evans Ave are stop controlled and are over 1200 ft away. No queue spillback expected. Few construction trucks/buses use the intersection. They should be adequately accommodated for by use of an apron.	Continue consideration
6	Traffic volume	Adequately catered for when a Single lane roundabout is used. The v/c ratio requirement is satisfied. (Use Charts 2A and 2C)	Continue Consideration. (Single lane roundabout adequate)
7	Safety	Roundabout is expected to improve safety since speed can be reduced to 20 mph and pedestrians will feel safer.	Roundabout is the best option if the budget will allow it
8	Cost	The earthworks required to correct the slope is a major cost component and a determining factor for the feasibility of the project. Initial estimates will be required to evaluate the cost comparison with other alternatives.	Roundabout is favored since it reduces vehicles speed

Case 3

Los Alto Parkway is a four-lane arterial in Sparks, Nevada. It intersects Pyramid way, Sparks Blvd and Vista Blvd (at two locations). Between Pyramid Way and Sparks Blvd there are ten streets that intersect with Los Alto Pkwy. Four of the critical cross streets are, Galleria Pkwy, Ion Drive, Promedio Pkwy and Table Mountain Way/Village Meadows Drive. Galleria Pkwy and Ion drive are controlled by traffic signals. Mountain way/Village Meadows Dr are right turn only because of a raised median curb. Residents and shoppers experience undue delays during the peak periods because of the volume of vehicle on the Los Altos Pkwy. There is the consideration to construct a series of roundabouts on Los Altos Pkwy between Pyramid way and Sparks Blvd on the four major intersections. The 2011 AADT for Los Altos Pkwy is 17,000 veh. The peak period traffic volumes measured for the intersection of Los Altos Pkwy and Gallaria Pkwy are 1250 and 325 vph respectively. For Los Altos Pkwy and Ion Dr intersection, the peak period volumes are 1300 and 180 vph respectively. The Tables 27-30 below are the answers to the questions when the flowchart is use.

For Village Meadows Dr/Table Mountain Way Intersection, the final decision is that, the intersection is NOT a good location for roundabout installation. This is because the likelihood of queue spillback is going to affect the operations. Promedio Pkwy, Ion Drive, Galleria Pkwy intersections are feasible locations for roundabout installation since all the factors seem to be appropriate. In conclusion, using minimal data and site visits, the decision flowchart shows that it is possible to explore the possibility of installing roundabouts at Promedio Pkwy, Ion Dr and Galleria Pkwy to reduce delays on the side street and improve safety as a whole for the Los Altos Pkwy between Pyramid way and Spark Blvd. A roundabout is however not recommended for Village Meadows Dr/Table Mountain Way.

Table 27 Los Altos Pkwy and Village Meadows Dr/Table Mountain Way Intersection (Case 3)

No	Consideration	Discussion	Roundabout Decision
1	Right of way	There appears to be enough ROW if it's a multi-lane with two lanes on Los Altos and single-lane on the cross street. It might require some additional land than currently being used since the residential houses are close to the streets.	Continue consideration
2	Grade	Relatively flat terrain so grade seems ok.	Continue consideration
3	Speed	Village Meadows Dr/Table Mountain Way are residential so have considerably lower speed than Los Altos Pkwy. Further speed reduction may be necessary on Los Altos.	Continue consideration
4	Pedestrian	Pedestrian volume is not expected to be an issue since currently there are less than 50 pph at the crossings. Chart 1A and 1B are used	Continue consideration
5	Roadway	The intersection is close to Sparks Blvd and queue spillback is a major concern at that intersection because of the volume of vehicles on both arterials.	Roundabout NOT recommended
6	Traffic volume		
7	Safety		
8	Cost		

Table 28 Los Altos Pkwy and Promedio Pkwy Intersection (Case 3)

No	Consideration	Discussion	Roundabout Decision
1	Right of way	The location has undeveloped land around it and so the imprint of the roundabout can be shifted around to adequately cater for the roundabout.	Continue consideration
2	Grade	The terrain is relatively flat. There will not be too much earthworks required since visibility is sufficiently good all round.	Continue consideration
3	Speed	The difference in speed is 5 mph which can be accommodated for at the roundabout with ease. The speed on the approach on Los Altos will need some reduction if necessary	Continue consideration
4	Pedestrian	Pedestrian volume is below 50 pph and should not adversely affect the capacity. Chart 1A and 1B are used	Continue consideration
5	Roadway	Since roundabout is not recommended for Village Meadows Dr/Table Mountain Way intersection, the possibility of queue spillback is unlikely.	Continue consideration
6	Traffic volume	Judging from the traffic volume on Ion Dr and Galleria Pkwy that have more traffic than Promedio, it is expected that roundabout should be adequate. Using Chart 2A and 2C, a multilane roundabout will be adequate. Double-lane on Los Altos Pkwy and Single-lane on Promedio Pkwy. The truck percentage is about 40 percent, but since a multi-lane is being recommended, it should take care of trucks.	Continue consideration
7	Safety	The main safety issue has to do with the left turning vehicles on the Promedio Pkwy. During the peak periods, they have to wait excessively long periods. A roundabout will reduce the waiting time since vehicles can accept smaller headways with vehicle slowed down.	Continue consideration
8	Cost	Considering the earthwork to be within normal cost range and the ROW sufficient enough, it is expected that the long term benefits from roundabout installation will outweigh that of signal. Roundabout is therefore a viable option which can be considered	Roundabout recommended

Table 29 Los Altos Pkwy and Ion Drive Intersection (Case 3)

No	Consideration	Discussion	Roundabout Decision
1	Right of way	Los Altos Pkwy has two lanes a short left turn pocket each direction. Ion drive has one lane each for through, left turn and right turn vehicles with two lanes for entering vehicle. The size of the intersection and the land around can sufficiently allow for the construction of a roundabout.	Continue consideration
2	Grade	The terrain is relatively flat. There will not be too much earthworks required since visibility is sufficiently good all round.	Continue consideration
3	Speed	The difference in speed is less than 10 mph which can be accommodated at the roundabout with ease. The speed on the approach on Los Altos will need some reduction if required	Continue consideration
4	Pedestrian	Pedestrian volume is about 100 pph and the effect is minimal on the capacity as well as the v/c ratio. Chart 1A and 1B are used to verify the effect on the capacity and deemed to be ok	Continue consideration
5	Roadway	The intersection is far enough from other intersections and so the risk of queue spillback is absent. If roundabouts will be installed on Galleria Pkwy intersection, there will not be the need for progression since the roundabouts will be able to reduce delays and allow vehicles to travel smoothly.	Continue consideration (Conditionally)
6	Traffic volume	Using Chart 2A, 2B and 2C, a multilane roundabout can adequately handle the traffic. Double lane on Los Altos Pkwy and Single-lane on the Ion Dr will sufficiently address the capacity needs on at the intersection. Since the truck percentage is less than 1 percent, it should be possible to have double circulatory lane to cater for the turning movement.	Continue consideration
7	Safety	Safety for both pedestrians and vehicles is expected to be enhanced with the construction of the roundabout since the speeds are expected to be slower.	Continue consideration
8	Cost	Since the ROW appears large enough, the additional land required if any is not expected to significantly increase the cost of construction. Also the grade is relatively flat so the earthworks required will not be too much. Roundabout is a feasible option at this intersection.	Roundabout recommended

Table 30 Los Altos Pkwy and Galleria Pkwy Intersection (Case 3)

No	Consideration	Discussion	Roundabout Decision
1	Right of way	Los Altos Pkwy has two lanes a short left turn pocket each direction. Galleria Pkwy has one lane each for through, left turn and right turn vehicles with two lanes for entering vehicle on the south bound direction. In the north bound direction, it has one lane each for through and right turn vehicles and two lanes for left turn vehicles and entering vehicles on the north bound. The size of the intersection and the land around can sufficiently allow for the construction of a roundabout.	Continue consideration
2	Grade	The terrain is relatively flat except for the east bound approach which might require some minor grade reduction if necessary. There will not be too much earthworks required since visibility is sufficiently good all round.	Continue consideration
3	Speed	The difference in speed is less than 10 mph which can be accommodated at the roundabout with ease. The speed on the approach on Los Altos will need some reduction if required	Continue consideration
4	Pedestrian	Pedestrian volume is about 100 pph and the effect is minimal on the capacity as well as the v/c ratio. Chart 1A and 1B are used to verify the effect and is acceptable	Continue consideration
5	Roadway	The intersection is far enough from other intersections and so the risk of queue spillback is absent. If roundabouts will be installed on Ion Pkwy intersection, there will not be the need for progression since the roundabouts will be able to reduce delays and allow vehicles to travel smoothly.	Continue consideration
6	Traffic volume	Using Chart 2A, 2B and 2C, a multilane roundabout can adequately handle the traffic. Double lane on Los Altos Pkwy and Single-lane on the Galleria Pkwy will sufficiently address the capacity needs on at the intersection. Since the truck percentage is less than 2 percent, it should be possible to have double circulatory lane to cater for the turning movement.	Continue consideration
7	Safety	Safety for both pedestrians and vehicles is expected to be enhanced with the construction of the roundabout since the speeds are expected to be slower.	Continue consideration
8	Cost	Since the ROW appears large enough, the additional land required if any is not expected to significantly increase the cost of construction. Also the grade is relatively flat so the earthworks required will not be too much. Roundabout is a feasible option at this intersection.	Roundabout recommended

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Nevada Department of Transportation
Rudy Malfabon, P.E. Director
Ken Chambers, Research Division Chief
(775) 888-7220
kchambers@dot.nv.gov
1263 South Stewart Street
Carson City, Nevada 89712