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Broadening Understanding of Roundabout Operation Analysis: Planning-Level Tools and Signal Application

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BROADENING UNDERSTANDING OF ROUNDABOUT OPERATION ANALYSIS:

PLANNING-LEVEL TOOLS AND SIGNAL APPLICATION.

by

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BROADENING UNDERSTANDING OF ROUNDABOUT OPERATION ANALYSIS:
PLANNING-LEVEL TOOLS AND SIGNAL APPLICATION

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In United States, roundabouts have recently emerged as an effective and efficient alternative to conventional signalized intersections for the control of traffic at junctions. This thesis includes two investigations related to the operations of roundabouts.

The first investigation examines the ability of a planning-level tool (the critical sum method) to serve as an indicator variable for the results of the Highway Capacity Manual's average delay per vehicle measure for a roundabout facility; to what extent do the results of one predict the results of the other? The critical sum method was found to accurately predict the HCM average delay per vehicle for low-volume conditions, approximately up to an average delay of 15 seconds per vehicle, but the tool was found to provide inaccurate predictions for higher volume conditions.

The second investigation looks at the potential of metering signals on a roundabout facility to transfer excess capacity from a low-volume approach to an adjacent higher-volume approach. The analysis indicated positive results for the theoretical benefits of the metering signal when only placing simulated traffic on two of the approaches, but the results were not duplicated when analyzing more-realistic volume scenarios with traffic on all four approaches.

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CHAPTER 1 INTRODUCTION

1.1. Background

Roundabouts were first used in the United Kingdom in the early 1900s, and have spread around the world since, dramatically increasing in number due to their ability to improve safety at intersections. The roundabout has also evolved in design since its inception, and is now an industry standard intersection design. Due to its yield control for vehicles entering the facility along with a reduced inscribed diameter, it improves both the safety and the capacity of an intersection over its larger-diameter predecessor known as the traffic circle or rotary. The modern roundabout mitigates safety concerns at a conventional signalized intersection through the reduction of both conflict points and speed, resulting in benefits to both crash frequency and severity. Allowing all traffic movements to move whenever they are not in conflict, the roundabout also decreases delay for low-volume conditions, relative to a conventional signalized intersection [1]–[6].

The primary performance metric for operations of a roundabout is the average delay per vehicle, most often estimated using the Highway Capacity Manual methodology [7], which is directly dependent on calculations of capacity, the number of vehicles per hour that can traverse the facility, and the ratio of the traffic demand that is present to the capacity. Secondary parameters are often considered in the course of an engineering study, and may include items such as expected accident rates, vehicle queue length, average number of stops per vehicle, vehicle emissions, and multi-modal considerations such as bicycle and pedestrian friendliness of the design.

The capacity of a given approach to a roundabout is inversely proportional to the number of vehicles circulating in front of that entry point. The greater the circulating flow, the fewer gaps there will be, and the shorter the gaps become, providing fewer opportunities for vehicles to pull into the circulating traffic, thus reducing the capacity [8]. Several methodologies exist for calculating the volume-to-capacity ratio of roundabout approaches. The critical sum method is a basic method that uses simple calculations for estimating the demand (of any intersection design), with an assumed constant capacity of 1,600 vehicles per hour per lane in conflict. The current edition of the Highway Capacity Manual (HCM) uses an exponential regression model based on gap acceptance for determining the capacity based on the circulating flow in front of an approach [7].

Roundabouts are typically designed with yield control on all approaches, providing priority for vehicles already within the circulating lane to avoid gridlock. However, the lack of signalization at the facility prevents engineers from changing the distribution of capacity from one approach to another, leading the modern roundabout design to become sensitive to negative performance on only one approach.

In a few isolated locations partial or full signalization of a roundabout has been attempted to manage unbalanced flow in conditions where traffic demands increased beyond expectations, but these have generally been stop-gap solutions and are not designed to be permanent fixes. Within consulting practice, it is generally the worst-case approach on a roundabout facility that will determine if it can be considered over the conventional signalized intersection alternative. There is an opportunity to design a signalization application to accommodate the types of demand flowrates where a facility is hindered by one approach performing worse than the others.

1.2. Research Questions

The objective of this research is to further explore roundabout operational analysis. This thesis seeks to answer two main questions. First, to what degree can the critical sum method provide a planning-level assessment of roundabout performance that is consistent with the results of the HCM formal analysis procedure for delay? Second, are there traffic conditions for which roundabout performance and safety can be improved through the use of signalization?

To answer the first question, a single-lane roundabout was analyzed to identify if the results of the critical sum method (CSM) calculation can serve as an accurate indicator variable for the subsequent results from HCM delay calculations. A total of 250,000 volume scenarios were examined, testing if knowledge of the value of the CSM for a given scenario was an accurate predictor of the average delay per vehicle for that same scenario.

To answer the second question, the impact of signalization of one movement of a roundabout facility was analyzed. This analysis focused on whether metering signals could successfully increase the gaps presented to a critical downstream approach, and if so, under what volume scenarios would they function, and what signal timing, if any, would be optimal.

1.3. Methodology

The primary analysis tools used to conduct the research included generated volume scenarios, planning level operational analysis tools, and VISSIM traffic microscopic simulation software.

To explore the reliability of using the critical sum method as a preliminary engineering tool for assessing the performance of a single-lane roundabout, the ability of the critical sum method value to serve as a predictor variable of the HCM average delay per vehicle was analyzed using a full-factorial experiment design including 250,000 volume scenarios. Sets of origin-destination volumes were generated, including different flow characteristics, such as balanced and unbalanced flow patterns, major and minor directional flows, and high and low turning movements.

Following these steps, the reliability of the critical sum method is calculated. Additionally, to indicate the maximum limit for this method to be effectively used, the percentage of data points having a value of average delay within an acceptable range from the observed mean delay was analyzed. **Figure 1-1** represents the summary of the methodology of this portion of the research.

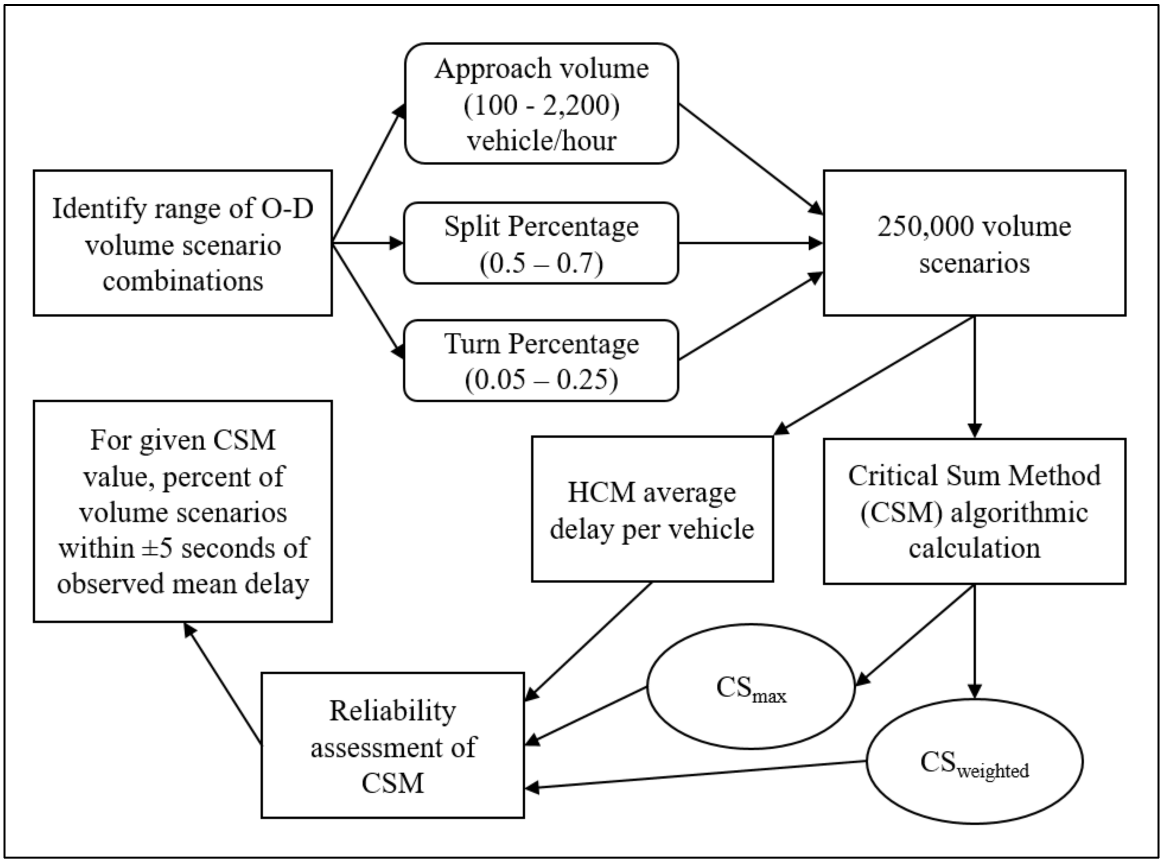


Figure 1-1 Research flow chart for assessing the critical sum method

To explore the impact of signalization on roundabout performance, a single-lane roundabout was modeled in VISSIM. Volume scenarios were generated to be generally realistic in terms of traffic demand flowrates, but also to create the intended situation with one approach experiencing higher demand flowrates than the other approaches at the facility. The approach experiencing the highest flowrate is considered to be the controlling approach, with the next approach immediately upstream from it the controlled approach, the one receiving signalization. First, the roundabout was analyzed with traffic loading only on the controlling and controlled approaches to explore the creation of artificial gaps within the circulating lane of the roundabout. Second, the roundabout was analyzed with traffic loading on all four approaches, with the controlling approach having more demand than the other three approaches. The signal timings were varied to examine the benefit, if any, to the average delay per vehicle due to the signalization of the controlled approach. The theory predicted significant increases in delay on the controlled approach, with decreases in delay to the controlling approach and the intersection overall, relative to the unsignalized alternative. **Figure 1-2** represents the summary of the methodology of this portion of the research.

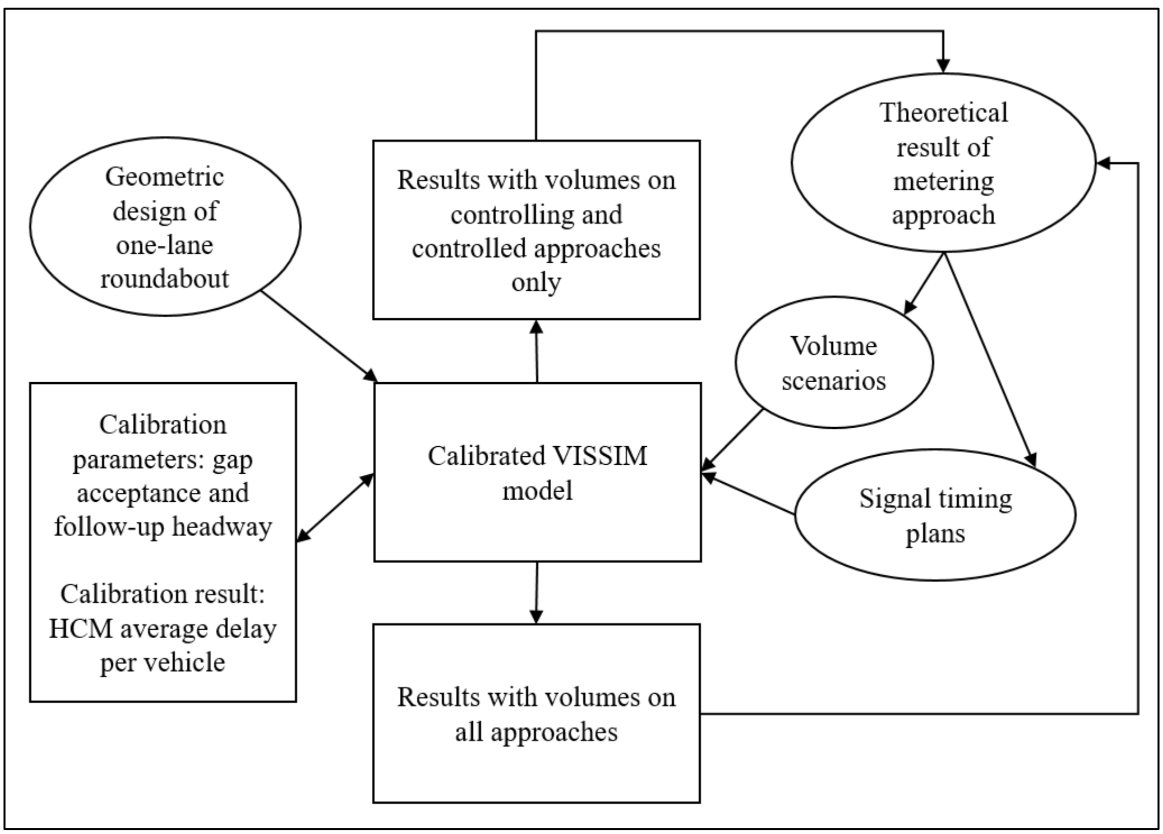


Figure 1-2 Research flow chart for assessing roundabout signalization

1.4. Document Layout and Attribution

This thesis is divided into five chapters. Chapter 1 is the introduction. Chapter 2 reviews the current state of the practice for roundabouts, examining their history, geometric design, safety considerations, operational performance measures and the different available methodologies for measuring the capacity of roundabouts, and signalization of roundabouts. Chapters 3 and 4 represent the body of this thesis, each chapter consisting of an individual paper which was published or submitted for conference presentation. Chapter 5 is the conclusion and recommendations for further research.

CHAPTER 2 ROUNDABOUT LITERATURE REVIEW

This chapter summarizes the important aspects of roundabout design and functionality to provide a foundational and contextual understanding for the research included herein**.**

2.1. General Information about Roundabouts

Circular intersections were developed in the 1900s to improve traffic flow and enhance safety. Throughout the history of circular intersections, several variations on their geometry have been developed. The first and largest iteration of circular intersections is the rotary, or traffic-circle, which has fallen from popularity due to safety concerns, and a propensity for the design to grid-lock in near-saturation conditions. More recently, modern roundabouts have been embraced as a standard alternative design to conventional signalized intersections.

2.1.1 History of Embrace in the United States

Columbus Circle in New York City was one of the first traffic circles built in the United States in 1905. Throughout the 1900s, traffic circles were part of the transportation system, but were reserved for somewhat unique situations such as the crossing of two rural highways in New Hampshire or the junction of more than four approaches in major metropolitan areas like New York City [5]. In 1960s, the modern roundabout was developed in the United Kingdom as an iteration of traffic circles, solving some of the key issues of the larger design. The issue of grid-lock in high-volume conditions was resolved by moving the yield/stop point from the interior of the circle to the exterior, freeing up the circulating lane, and the issue of safety was resolved by forcing traffic to slow while maneuvering smaller-radii curves, and by a drastic reduction in conflict points relative to the conventional signalized intersection [3]. This new design started gaining interest in the 1990s in the United States, and its safe, efficient, and cost effective design has become commonplace throughout the country [8].

2.1.2 Geometric Design

The first-come-first-serve nature of yield control for vehicles entering the roundabout resolves the question of conflict resolution in the absence of signalization. The primary goal of geometric design at these facilities is then to aid (or force) drivers to slow their speeds down while navigating the facility, with slow speeds being a key component of the safety benefits experienced at these facilities. Other considerations that impact the design of a roundabout are the site constraints, speed of roadways approaching, requirement to pass heavy vehicles, and various geometric attributes unique to each individual site [8]. Three fundamental elements must be determined in the preliminary design stage of a roundabout before defining the details of the geometry: the optimal roundabout size, position, and alignment and arrangement of approach legs [1]. Some of the basic geometric design elements for a roundabout are shown below in **Figure 2-1**.

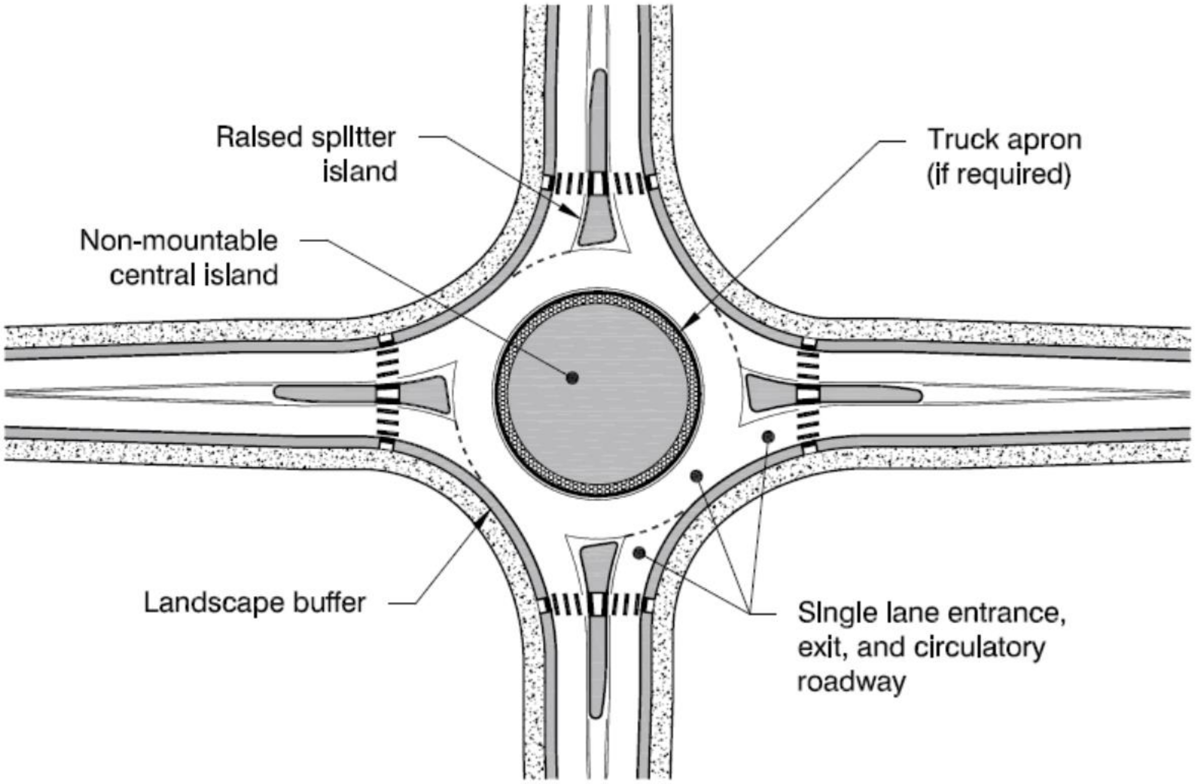


Figure 2-1 Basic geometric elements of a roundabout [1]

Roundabouts have different characteristics based on location, with rural and urban areas imparting different needs upon a facility. For example, at rural roundabouts, typical bicycle and pedestrian volumes are low, which enables the use of higher speed approaches than could be used in urban locations. That being said, it is imperative to assess the needs of each specific location, which prevents the use of things like standard details for urban or rural locations.

Roundabouts are divided into three main types based generally on the size of the facility, as a function of how many vehicles it services, with the three types being mini roundabout, single lane roundabout, and multilane roundabout [8].

2.1.2.1 Mini Roundabout

To overcome the safety problems and excessive delays on minor approaches at physically constrained intersections, a mini roundabout can be used as an alternative design, as shown below in **Figure 2-2**. Mini roundabouts are appropriate in locations where the approaching roadways have an 85th percentile speed less than 30 mph (50km/h), and in some parts of the United States are beginning to be used extensively within subdivisions. Mini roundabouts are discouraged at locations with excessive U-turn traffic [5].

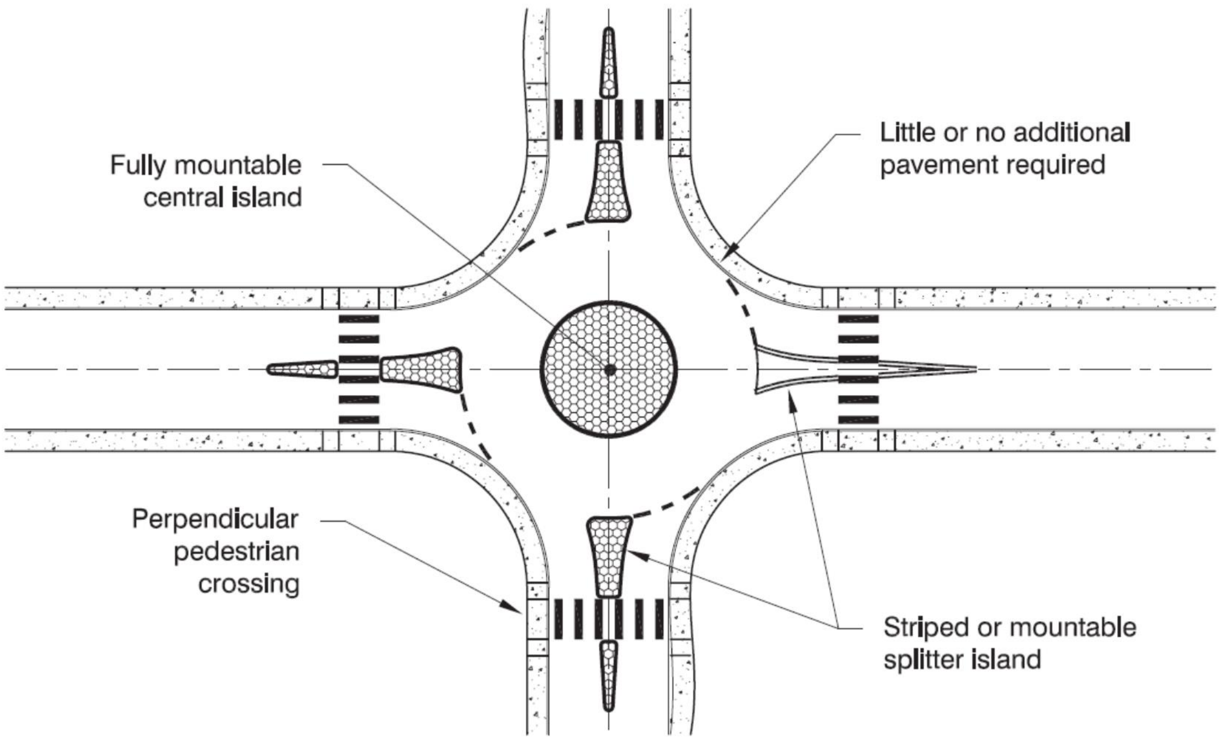


Figure 2-2 Example of a mini roundabout [8]

2.1.2.2 Single Lane Roundabout

At single lane roundabouts, the circulatory roadway should be designed with a minimum radius to accommodate the design vehicle, avoiding larger radii in order to maintain slow speeds in the facility. Moreover, it is common to provide a mountable curve on the interior of the roundabout with a drivable pad that allows heavy vehicles a straighter path through the facility while maintaining the requirement of passenger cars (and equivalent) to be diverted at slower speeds than they would be traveling straight through. Appropriate vehicle turning templates or a CAD-based computer program should be used to determine the swept path of the design vehicle through each of the turning movements. Usually the left turn movement is the critical path for determining circulatory roadway width. In accordance with AASHTO policy, a minimum clearance of 2 ft (0.6 m) should be provided between the outside edge of the vehicle's tire track and the curb line. AASHTO Table III-19 (1994 edition), shown below as **Table 2-1**, provides derived widths required for various radii for each standard design vehicle [5].

Table 2-1 Recommended inscribed circle diameter ranges [5]

Site Category	Typical Design Vehicle	Inscribed Circle Diameter Range*
Mini-Roundabout	Single-Unit Truck	13–25m (45–80 ft)
Urban Compact	Single-Unit Truck/Bus	25–30m (80–100 ft)
Urban Single Lane	WB-15 (WB-50)	30–40m (100–130 ft)
Urban Double Lane	WB-15 (WB-50)	45–55m (150–180 ft)
Rural Single Lane	WB-20 (WB-67)	35–40m (115–130 ft)
Rural Double Lane	WB-20 (WB-67)	55–60m (180–200 ft)

* Assumes 90-degree angles between entries and no more than four legs.

2.1.2.3 Multiple Lane Roundabout

At multi-lane roundabouts, the width of lanes within the circulatory roadway is usually governed by light trucks and not the design vehicle's overhang while turning, as the design vehicle is assumed to utilize the mountable curb and adjacent area to perform their maneuver. The width required for the necessary number of adjacent lanes to travel simultaneously through the roundabout should be used to establish the circulatory roadway width. The combination of vehicle types to be accommodated side by side is dependent upon the specific traffic conditions at each site. If the entering traffic is predominantly passenger cars and single unit trucks (AASHTO P and SU vehicles), where semi-trailer traffic is infrequent, it may be appropriate to design the width for two passenger vehicles or a passenger car and a single unit truck side-by-side. If semi-trailer traffic is relatively frequent, such as being greater than 10 percent, it may be necessary to provide sufficient width for the simultaneous passage of a semitrailer in combination with a passenger vehicle or single unit truck [8]. Examples of two- and multi-lane roundabout geometries are shown below, in **Figure 2-3** and **Figure 2-4**.

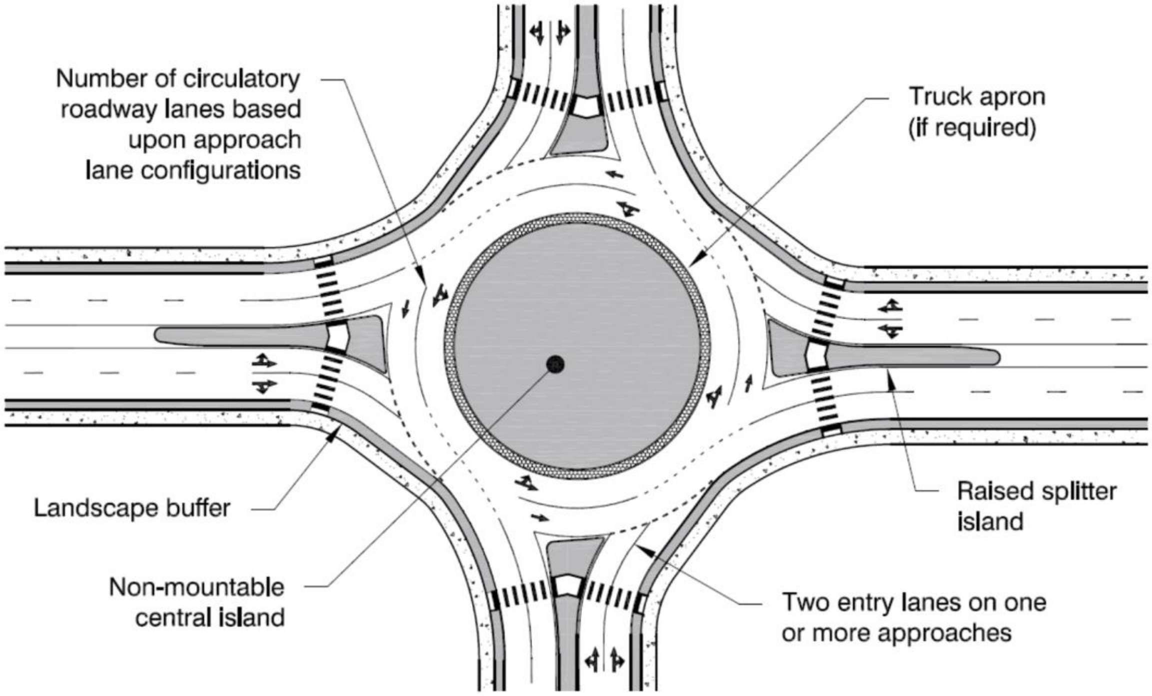


Figure 2-3 Example of a two-lane roundabout [8]

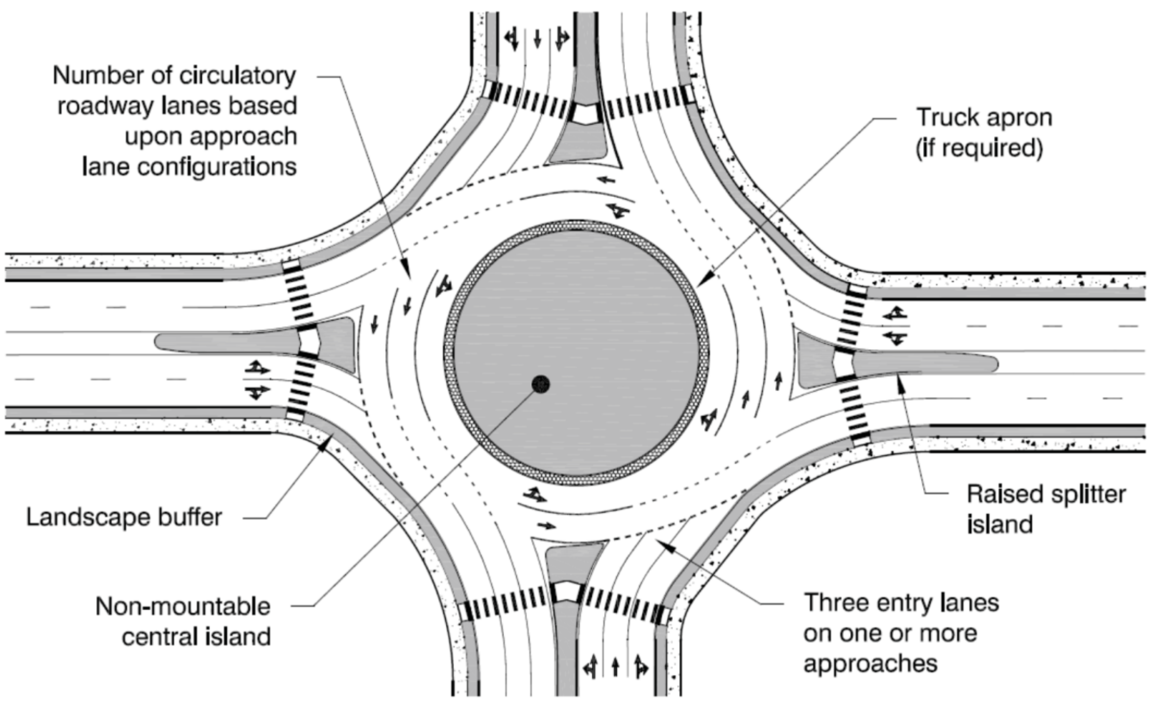


Figure 2-4 Example of a three-lane roundabout [8]

2.1.3 Safety Considerations

One of main advantages of using roundabouts is to enhance the safety of intersections [5], [1], [6]. In comparison to conventional intersections, roundabouts have decreased number of conflict points. Also, the geometry helps to reduce high-severity conflicts such as left turn, head on, and right angle crashes. **Figure 2-5** provides a comparison of conflict point diagrams for a roundabout and a conventional intersection. The curve radii along the traversed path of the vehicle forces drivers to reduce their speeds upon entering a roundabout, which enables both the circulating and entering vehicles to have more time to react to the conflict points with the facility. Speed reduction leads to a decrease in the fatal and serious injuries, as less damage is done in the unfortunate case of a collision. Traversing a roundabout, pedestrians are only required to cross one direction of traffic at a time at each approach. However, the lack of a protected pedestrian crossing phase, such as what can be found at a signalized intersection, increases the danger to pedestrians, particularly those who have a vision impairment [1], [6]. High pedestrian volumes at a junction, particularly if there is a known presence of vision impaired pedestrians, may be a sufficient reason to reject a roundabout as a potential design solution at a location. Properly designed roundabouts will additionally limit the speed change as drivers navigate complex and reverse horizontal curves along their travel path, compared with a traditional signalized intersection, which minimizes single-vehicle crash rates and severity.

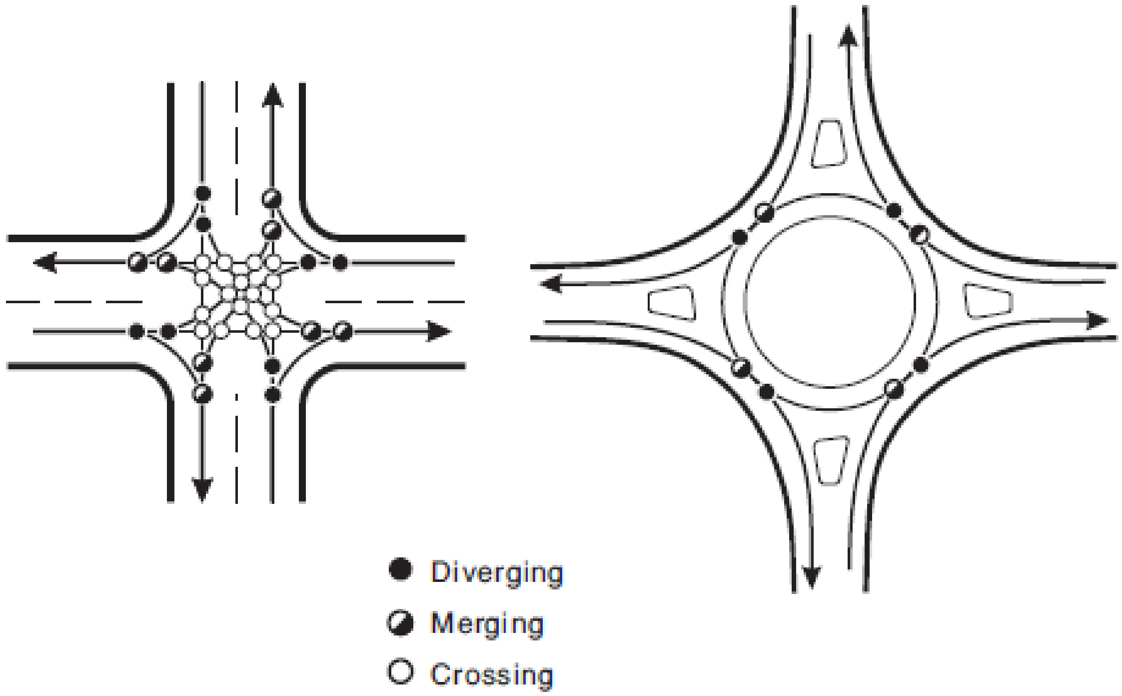


Figure 2-5 Conflict points for conventional intersections and roundabouts [9]

2.2. Operational Performance of Roundabouts

The primary performance metric used within the United States for roundabout operations is the average delay per vehicle, which is calculated as a function of capacity and (demand) volume-to-capacity ratio, with capacity being directly influenced by the gap acceptance and follow-up headway behaviors of drivers.

2.2.1 Measures of Capacity

The HCM defines capacity as "the maximum hourly rate at which persons or vehicles can be reasonably expected to traverse a point or a uniform segment of a lane or roadway during a given time period, under prevailing roadway, traffic, and control conditions" [10].

Capacity at a roundabout facility is usually measured at the entry of each approach to the facility, referencing the various flows that are entering (v_e), exiting (v_{ex}), and circulating (v_c) as shown in **Figure 2-6**, below.

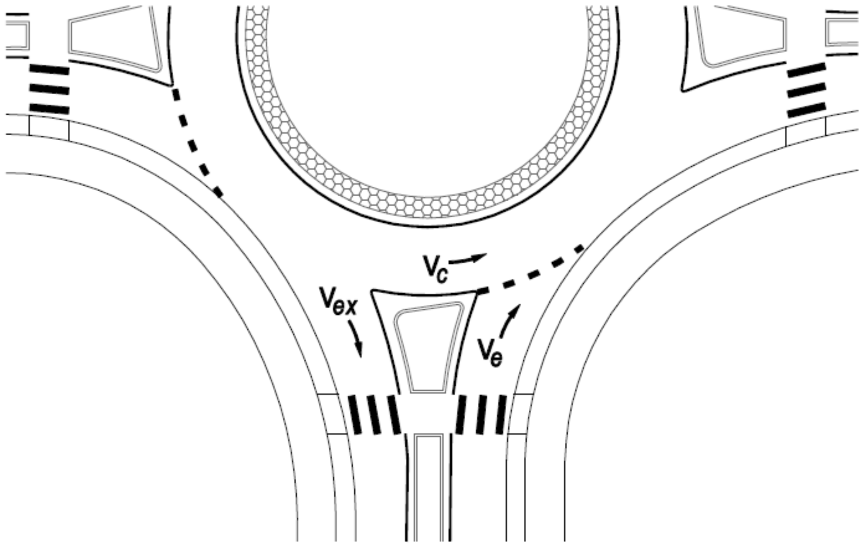


Figure 2-6 Volumes associated with roundabout approach [11]

An example calculation of the circulating flow is shown below by way of *equation 2-1* and **Figure 2-7**.

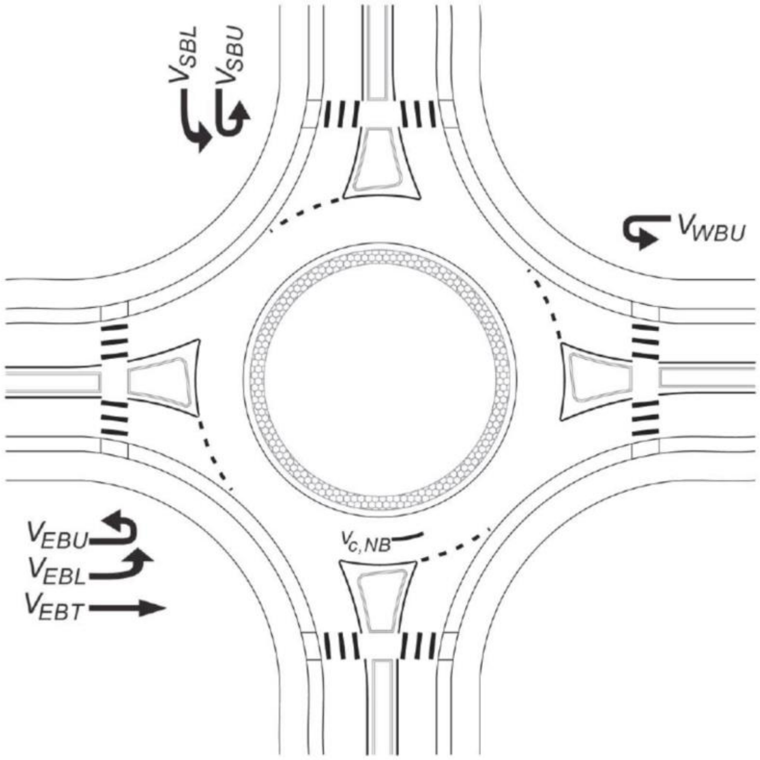


Figure 2-7 Calculation of circulating flow [7]

$$v_{c,NB} = v_{WBU} + v_{SBL} + v_{SBU} + v_{EBT} + v_{EBL} + v_{EBU} \qquad \textit{Eq. 2-1}$$

$v_{c,NB}$ = the circulating flow rate for the northbound (veh/hr)

v_{WBU} = the flow rate for the westbound u-turn (veh/hr)

v_{SBL} = the flow rate for the southbound left (veh/hr)

v_{SBU} = the flow rate for the southbound u-turn (veh/hr)

v_{EBT} = the flow rate for the eastbound through (veh/hr)

v_{EBL} = the flow rate for the eastbound left (veh/hr)

v_{EBU} = the flow rate for the eastbound u-turn (veh/hr)

Circulating flow (v_c) is utilized as the sole parameter in the HCM methodology for determining the capacity of an approach. The higher this circulating flow is, the fewer and smaller the gaps are that are presented to the entering vehicles, and the lower the capacity and greater the delay of the approach [5], [8], [12]. Besides the number of lanes on an approach, other geometric elements, for example the width of the lane on the approach at the point of entry to the circulating lanes or the angle of entry, are utilized by some methodologies, most notably RoDel. The research supporting the HCM methodology looked into these geometric factors in the dataset collected to develop the U.S.-based model, but did not find these factors to play a significant role in determining capacity based on driver behavior within the United States [11].

It is the size and number of gaps presented to vehicles on an approach, combined with their willingness to accept these gaps, which ultimately determines the capacity of a roundabout approach. The calibration parameters most often used in microsimulation software to model gap acceptance behavior of drivers are the critical headway (also known as the critical gap) and the follow-up headway.

2.2.1.1 Critical Headway

Critical headway (or critical gap) is the minimum time required by a driver to accept a gap presented to them and safely enter a roundabout. According to Tian et al [13], critical headway may vary by location, and is also influenced by local and regional changes in driver behavior, geometric layout, and traffic conditions. One measure of this parameter in the field comes from the NCHRP 572 report, which determines critical headway as greater than the largest rejected headway and less than the accepted headway by the driver, as shown below in **Figure 2-8**.

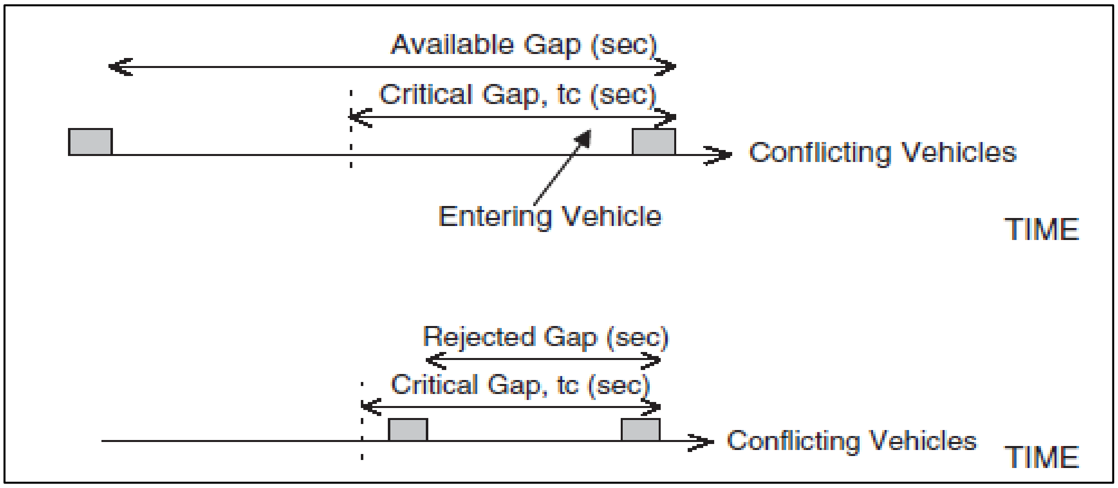


Figure 2-8 Accepted and rejected gaps in critical headway [11]

There is variety in the literature regarding the best way to devise the critical headway using observed gaps from field data [14]. The first method was a mathematical model called the maximum likelihood method [15]. It assumed the critical headway of a single driver ranges between the driver's accepted headway and largest rejected headway and estimates the average critical headway based on these. The second method was a statistical method known as the median method [16]. Here the critical headway was estimated as the average between the largest rejected headway and the accepted headway. The third method was a graphical method that was developed by Raff where the critical headway was considered as the intersection between the accepted gap lengths and the rejected gap lengths [17], [18]. This study indicated that the estimated critical headway between the three methods varied slightly. However, the results of the first and second method were most similar and the third method provided smaller critical headway values based on the different sites the authors tested.

2.2.1.2 Follow-up Headway

Follow-up headway is considered as the minimum headway between two successive vehicles (in the same lane) entering the roundabout from a given approach. According to Xu et al. [19], the follow-up headway can be estimated by taking the average difference between the passage times of two entering vehicles accepting the same gap in a conflicting stream under a queued condition.

The latest edition of the HCM incorporated the study done by Tian et al which calibrated the critical and follow-up headways of the capacity models based on field observation and site analysis done in California [20]. It was observed from this field study that for single lane roundabouts, the critical headway ranged between 4.4 and 5.3 seconds, leading to a mean value of 4.8 seconds. Similarly, for multi-lane roundabouts the critical headway ranged between 4.4 and 5.1 seconds for the left lane and 4.0 and 4.8 seconds for the right lane. An average value of 4.7 and 4.4 seconds were considered for the model. These values did not show any statistical difference from the critical headways of research sites included in NCHRP 572, primarily in Maine, Maryland, and Washington states [11]. Likewise, the follow-up headway was found to have a mean value of 2.5 seconds for single-lane roundabouts and 2.2 seconds for both lanes of a two-lane roundabout, a reduction from the previously calibrated values in NCHRP 572 of 3.2 and 3.1 seconds, respectively [20].

According to the observations conducted from the field studies as reported from NCHRP 572, critical headway and follow-up headway were indicated to have a moderate to low negative correlation with the conflicting flow rate and speed [11].

2.2.1.3 Critical Sum Method

The Critical Sum Method (CSM) is an approach for calculating the capacity of a junction through basic calculations, and can be applied to all intersection and interchange designs, both signalized and unsignalized, without first determining the signal timing. The CSM calculates a measure of demand for signalized intersections by calculating the demand flowrates of each movement in terms of vehicles per hour per lane, and then moving through the signal phase diagram and summating all of the critical movements to get a total peak hour demand flowrate. When applied to roundabouts, the critical sum method is calculated separately to each approach, summating the entering flow (v_e) and the circulating flow (v_c). The capacity against which this demand is compared is taken by default as equal to 1,600 vehicles/lane/hour independent of which type of intersection is being considered [21], with only one study to date examining the assumption of whether all intersection designs reach capacity at the same critical sum value [22].

Presenting this method by way of equations, the value of critical sum on each approach (i) of a roundabout (CS_i) is found based on the calculation shown in *equation 2-2*, and sums the conflicting flow ($v_{c,i}$) with the entering flow ($v_{e,i}$) on the approach.

$$CS_i = v_{e,i} + v_{c,i} \qquad \textit{Eq. 2-2}$$

$$for\ i =$$

$$eastbound, northbound, westbound, and\ southbound\ approaches$$

$v_{e,i}$ = the demand flowrate on approach *i* (veh/hr)

$v_{c,i}$ = the circulating flowrate passing in front of approach *i* (veh/hr)

An application of the critical sum method has been used by the Federal Highway Administration to develop the excel-based software called Capacity Analysis for Planning of Junctions (Cap-X), incorporating nine intersection and six interchange designs, including recently developed alternative intersections and interchanges [23]. A previous study used this software to explore the theoretical benefits of the various alternative designs [24].

2.2.1.4 Roundabout Capacity Calculations in the Highway Capacity Manual

The HCM implements the research conducted in NCHRP 572 and calculates the capacity of roundabout approaches using an exponential regression equation that assumes a relationship between circulating flow and capacity that is independent of other factors such as design geometry of the facility. Different values are calibrated for the exponential equation for single-lane roundabout approaches, and the individual lanes of each approach for multi-lane roundabout approaches.

2.2.1.4.1 Single Lane Roundabouts

The equation used in the sixth edition of the HCM [8] for calculating the entering capacity of a given approach on a single-lane roundabout is dependent upon the conflicting flow as presented in *equation 2-3*.

$$c_{e,pce} = 1380 * exp^{(-0.00102 * v_{c,pce})} \qquad \textit{Eq. 2-3}$$

$c_{e,pce}$ = lane capacity, adjusted for heavy vehicles (pc/h), and

$v_{c,pce}$ = conflicting flow rate (pc/h).

2.2.1.4.2 Multi-Lane Roundabouts

There are several configurations of entry lanes and conflicting lanes that could exist for a multi-lane roundabout approach. Independent equations were developed for each individual lane set or lane combination for estimating the capacity of the roundabout. The calculations reflected the observations in a completed field study [8], [25]. The calculations are represented in *equations 2-4* through *2-7*.

Capacity for Two Lane Entries Conflicted by One Circulating Lane

$$c_{e,pce} = 1{,}420 * exp^{(-0.00091 * v_{c,pce})} \qquad \textit{Eq. 2-4}$$

Capacity for One Lane Entries Conflicted by Two Circulating Lanes

$$c_{e,pce} = 1{,}420 * exp^{(-0.00085 * v_{c,pce})} \qquad \textit{Eq. 2-5}$$

Capacity for Two Lane Entries Conflicted by Two Circulating Lanes

$$c_{e,R,pce} = 1{,}420 * exp^{(-0.00085 * v_{c,pce})} \qquad \textit{Eq. 2-6}$$

$$c_{e,L,pce} = 1{,}350 * exp^{(-0.00092 * v_{c,pce})}$$ *Eq. 2-7*

$c_{e,R,pce}$ = capacity of the right entry lane, adjusted for heavy vehicles (pc/h),

$c_{e,L,pce}$ = capacity of the left entry lane, adjusted for heavy vehicles (pc/h),

$v_{c,pce}$ = conflicting flow rate (total of both lanes) (pc/h).

Figure 2-9 shows the plot of the capacities for the above equations' single-lane and multi-lane roundabouts.

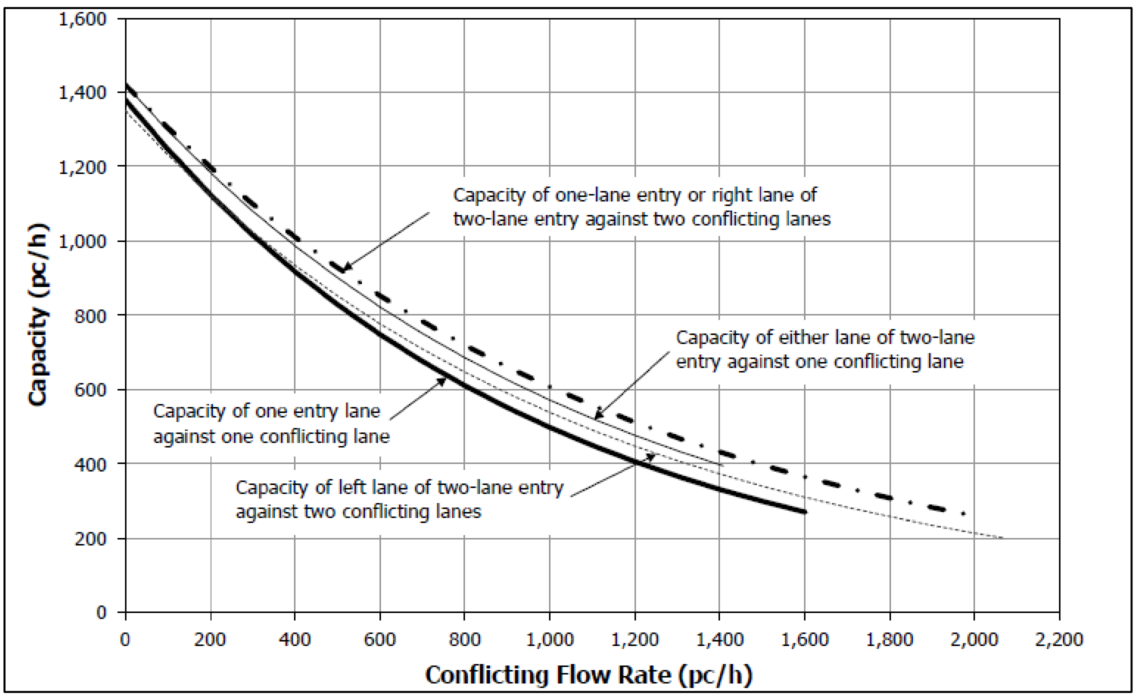


Figure 2-9 Capacity of single-lane and multi-lane entries [8]

In developing the regression models for the capacity equations shown as *equation 2-3* through *equation 2-7*, a range of data points was collected, as shown below in **Figure 2-10** [11]. In the case of single-lane roundabouts, circulating flows between 0 and approximately 1,000 vehicles per hour were used to calibrate the model. The observed maximum entry flow for a single-lane roundabout ranged between 500 and 1,400 with minimal conflicting flow, while the observed maximum entry flow against 900 vehicles conflicting ranged from 200 to 500. Interestingly, the intercept for the calibrated model from **Figure 2-9** is at 1,380 with no conflicting flow, and the model is shown extending out to a conflicting flow of 1,600, corresponding to an entry capacity of around 270. This data serves as a cautionary warning that the experiences of an individual facility may vary widely from the prediction of the best-fit model.

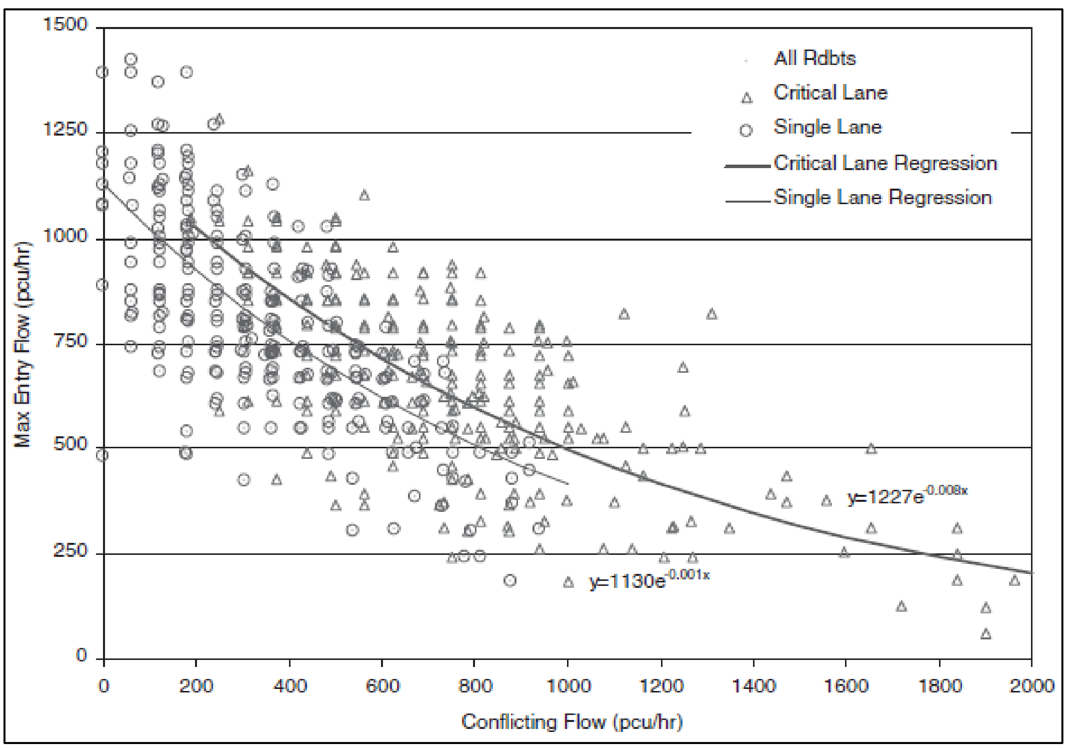


Figure 2-10 Single-lane and multilane critical-lane regression [11]

2.2.2 Measures of Delay

The sixth generation of the Highway Capacity Manual uses average delay per vehicle as the primary measure of effectiveness for both signalized and unsignalized intersections. However, only the control delay is included in the HCM, which includes the time taken by a driver to decelerate to a queue, queue, wait for an acceptable gap in the circulating flow at the front of a queue, and to accelerate out of a queue is usually considered to be the control delay. The control delay measure notably excludes the additional travel time imparted by the geometry of the junction, which causes great difficulty when endeavoring to compare alternative geometries against one another [26]. The HCM delay calculation is presented in *equation 2-8* [8].

$$d_i = \frac{3600}{c_i} + 900T\left[x_i - 1 + \sqrt{(x_i - 1)^2 + \frac{\left(\frac{3600}{c_i}\right)x_i}{450T}}\right] + 5 * min[x_i, 1] \qquad \textit{Eq. 2-8}$$

$for\ i =$

$eastbound, northbound, westbound, and\ southbound\ approaches$

d_i = average control delay for approach *i* (seconds/vehicle)

x_i = volume-to-capacity ratio of approach *i* (unitless)

c_i = capacity of approach *i* (vehicles/hour), and

T = analysis time period (hour)

As observed from *equation 2-8*, average control delay for a given approach is a function of lane capacity and degree of saturation. The HCM assumes no residual queue at the initial period of the analysis. Figure **2-11** presents the variation of control delay as a function of the capacity and entering flow.

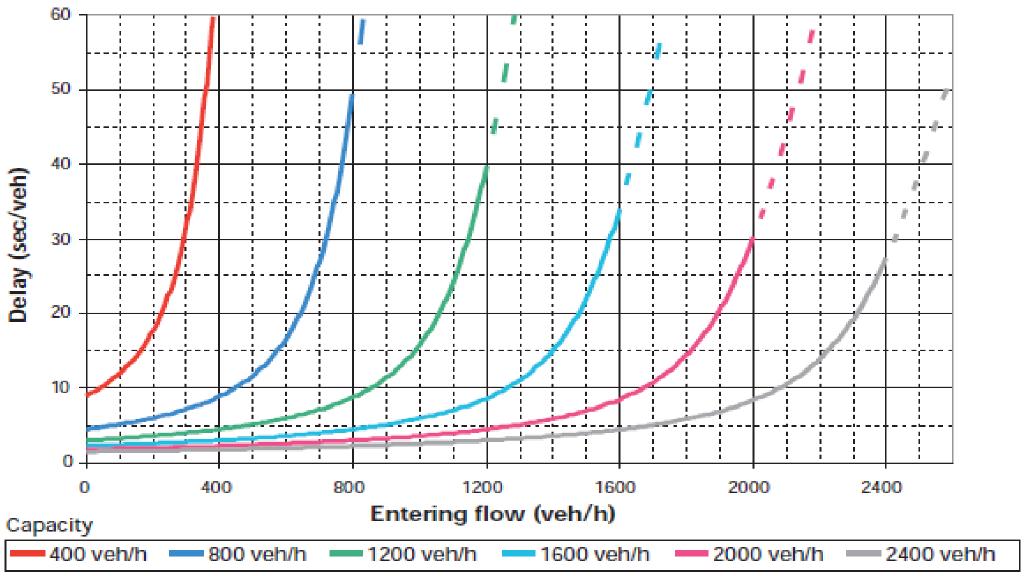


Figure 2-11 Control delay as a function of capacity and entering flow [8]

Another way of conceptualizing the control delay experienced at a roundabout is the difference between the desired speed of a driver if the facility were not present, compared against the actual speed of the driver as they pass through the facility. An example of the speed of drivers as they pass through a roundabout facility is shown below in **Figure 2-12** [4].

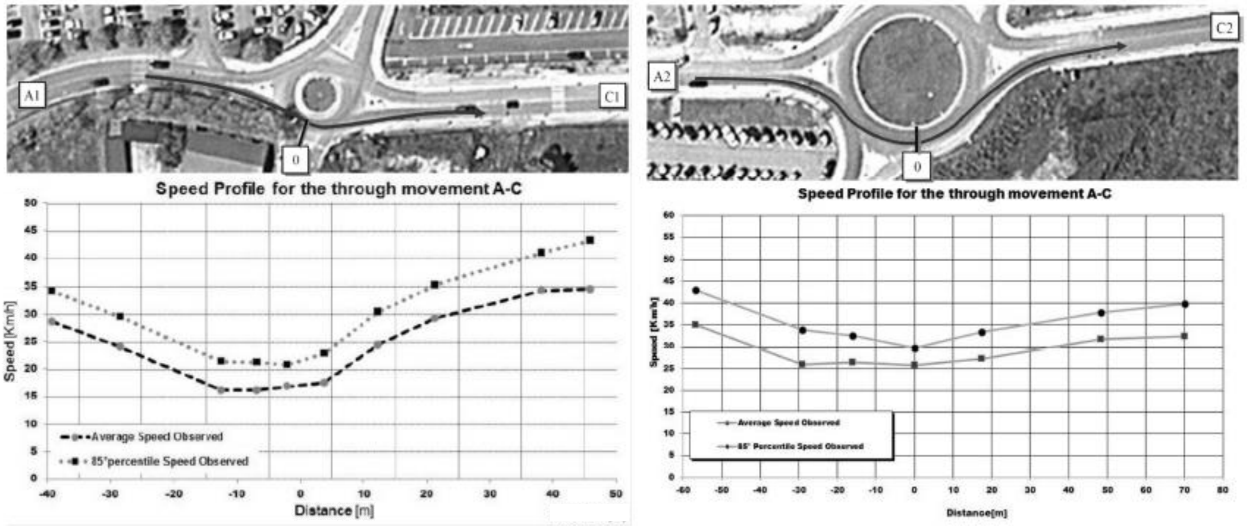


Figure 2-12 Speed profiles for the through movements at roundabouts [4]

2.2.3 Use of Volume-to-Capacity Measures as an Indicator of Delay

The sixth edition of the HCM uses control delay as the primary measure of effectiveness for both signalized and un-signalized intersections. However, the HCM method for calculating delay is complex to the point of requiring computer software to compute, particularly in the case of signalized intersections which require detailed information about signal timing to calculate, often impractical for purposes of preliminary design engineering. The Critical Sum Method (CSM), also known as the critical lane analysis method, provides a simplified analysis procedure that can be conducted for all intersection geometries, both signalized and unsignalized, knowing only the number of lanes per lane group, the turn movement volumes, and a conceptual schema of the signal phasing at the facility, or in the case of a roundabout, the entry and circulating flowrates. With a simple mental calculation, the critical sum method can provide an indication of performance of a facility at a preliminary stage of design. It is used widely in consulting engineering for preliminary estimates of facility operations, such as when sitting down with a client for the first time, as well as an analysis tool to identify problematic facilities within a large system. However, very little information exists in the research regarding the reliability of the CSM as an indicator variable for the average delay results that are calculated with more complex methodologies [27]. Particularly in the case of roundabout facilities that don't require assumptions about signal timing, it is important to assess whether CSM is a suitable indicator variable of HCM average delay calculations.

2.2.4 Other Analysis Methods Commonly in Use

Other than the HCM methodology for calculating average delay per vehicle at roundabout facilities, the primary software applications (and corresponding methodologies) that were developed abroad and are implemented in practice in the United States are SIDRA from Australia and RoDel from the United Kingdom.

2.2.4.1 SIDRA

SIDRA is a popular software application developed in Australia, and widely used in consulting within the United States for analyzing roundabout operations. The gap acceptance parameters for the model implemented within SIDRA are calibrated from several surveys conducted at modern roundabout facilities in Australia. The methodology for calculating capacity in SIDRA is similar to that implemented by the HCM, and examines gap acceptance and follow-up headway as the primary input variables. One variation in the capacity model implemented in SIDRA is that it is sensitive to approach queues, with gap acceptance values decreasing as wait time increases for approaching vehicles, modeling the aggressive responses of drivers to higher wait times [28]–[30], [31].

2.2.4.2 RoDel (Roundabout Delay)

In terms of the number of input parameters, the most comprehensive modeling of roundabout performance for models used within the United States comes from Roundabout Delay (RoDel), a software package developed in the United Kingdom (UK). RoDel calibrates entry capacity using driver behavior parameters such as gap acceptance and follow-up headway, as well as making extensive use of the geometric design of the facility, such as the angle of an approach and the width of the circulating lane. With the aid of several surveys on both large and small roundabout facilities within the UK, RoDel as a model has been found to provide accurate results for the behavior of drivers specifically on facilities within the UK [29], [30], [32]. However, some studies indicate the failure of the RoDel model to accurately reflect performance of roundabout facilities within the United States, particularly when dealing with congested conditions [33],[34],[35],[36].

2.3. Signalization of Roundabouts

Roundabouts are designed to operate with yield control on the entering approach, providing priority movement to the circulating lane(s), to avoid gridlock of the facility when saturated flow conditions are reached. However, lacking the ability to prioritize one movement over another by way of signal timing, roundabouts are susceptible to poor operations due to a single approach experiencing high demand flowrates.

2.3.1 Roundabout Reduction in Service from Unbalanced Flow

During commuting traffic in the a.m. and p.m. peak periods, surface roadway networks routinely encounter directional flow patterns caused by prevailing traffic that moves from home to work in the morning and from work to home in the afternoon. At roundabout facilities, which are sensitive to imbalances in traffic flow [37], this directional flow can cause one approach to experience high delays, reducing the operational performance of the entire facility despite satisfactory performance by the other approaches. A signalized intersection could be implemented at the site, which would allow signal timing to redistribute the total capacity at the junction from lower-demand approaches to higher-demand ones, but doing so would lose the safety and cost benefits associated with the roundabout [38]. In rare cases, a few roundabout locations have been signalized as a stop-gap solution, when the facility was no longer functioning adequately but reconstruction of the site as a conventional signalized intersection replacement was not yet available.

2.3.2 Previous Experience with Signalization of Roundabouts

Full signalization, with all four approaches signalized in a similar fashion to a conventional signalized intersection, has been used to control the circulating traffic at junctions where existing roundabout facilities experience significant increases in traffic demand, though these implementations have been at the larger traffic-circle or rotary type designs thus far and not at a smaller radius modern roundabout [5], [39]–[41]. In most cases a queue detector is installed for deciding the optimal time for activating the signal, however, the literature indicates that fixed time-of-day signal timing has also been used [37], [39], [42], [43].

The most common solution found in the literature is to install signals at selected approaches of the roundabout when heavy directional demand exists during peak hours. In these situations, two methods have been used for controlling a roundabout, either with full signalization of the facility, or with a signal on a single approach. Alternatively, one study examined an attempt to mitigate a poor-performing roundabout by adjusting the timing on a nearby signalized intersection, but this was shown to be less effective than direct signalization of the roundabout [44]. **Table 2-2** presents six different ways indicated by Hallworth [39] and formalized into a table by Natalizio [38] for introducing signal control at a roundabout[39], summarizing the different methods thus-far used.

Table2-2 Types of signal control at roundabouts [38]

Design Parameter	Option	Description
1. Means of Control	Direct	Signal control of external and internal approaches (i.e. all conflict points are controlled)
	Indirect	External traffic only controlled by signals some distance from the entry point (to meter entry flow); circulating traffic is not controlled and therefore has priority
2. Full / part-time	Full	Signals permanently operating
	Part	Part-time only, switched on by time switch or queue detectors
3. Full / part-control	Full	All approaches with signal control
	Part	One or more approaches remain under priority control

Hallworth’s taxonomy of signalized control for roundabout facilities includes the means of control, whether the signal is operated part time or full time, and whether full control is implemented (on every approach), or if only some approaches are signalized. A facility experiencing direct control will have signalized control applied at the entry points to the circulating lane, while a facility utilizing indirect control will use upstream signalization, either mid-block or at an adjacent signalized intersection, to attempt to control the demand flowrate for one or more approaches. A signalized roundabout with part-time signalization may use time-of-day programming to determine when the signal is used, or may implement actuated control with detectors determining when the criteria are being met to justify use of the signal. Finally, full-signalization is often implemented as a partial fix before a facility is reconstructed as a conventional signalized intersection, while partial-signalization controls only some approaches at the facility, and has been used as a longer-term solution.

Within the English-language literature, examples of roundabout signalization have been found in Australia, the United Kingdom, and the United States of America. The primary purpose of these implementations has been to avoid excessive queuing and delays at the approaches affected by high directional flows [39], [43]. A study done by Stevens briefly reported some of implemented signalized roundabouts as presented in **Table 2-3** [43].

Table 2-3 Examples of signalization of roundabouts [43]

Roundabout	Means of control	Time of operation	Approach control	Problem areas
Charles Street, MD, USA	Indirect	Full	All	Unbalanced flow
Clearwater, FL, USA	Indirect	Full	One	Unbalanced flow during spring break
Penn Inn, Abbot, UK	NA	Full	All	Unbalanced flow and excessive turning movements

Unfortunately, the only two documented cases of roundabout signalization within the United States are one case in Maryland and one in Florida. The site in Clearwater, Florida [42] features a signal actuated by a queue detector, installed on the highest-volume approach to create gaps for downstream traffic. At this location, the high-flow approach consistently prevented entrance to the facility by the downstream approaches, causing their queues to back up into adjacent intersections during the peak period of the day. At the Charles Street facility in Maryland, a traffic signal was installed because two legs of the roundabout represented over 90% of the flow, and it was deemed necessary to prioritize capacity for the higher-volume approaches over the lower-volume ones.

2.3.3 Safety Considerations for Signalizing Roundabouts

There are also safety impacts which may result from the signalization of a roundabout. Implementing a traffic signal at a roundabout approach has been found to reduce the circulating speed observed at the facility, which improves safety in terms of the weaving and merging process [39]. Additionally, signalizing a roundabout may increase the safety of pedestrians, potentially providing a dedicated pedestrian phase.

The Highway Safety Manual (HSM) uses Crash Modification Factors (CMF) to indicate the effect of implementing a treatment or a design. A study done by Richard provides documentation of a reduction in vehicle crashes and injuries in roundabouts when compared to regular conventional intersections [45]. However, the literature related to CMFs does not include any studies examining the impact of signalizing roundabouts [7].

2.3.4 Recommendations for Signal Control

Recent studies completed in the early and mid-2000s regarding roundabout signalization mainly highlighted two areas needing further investigation [44], [38].& [43]. The first is the impact of the signalization of a roundabout on driver behavior; are shorter gaps and shorter queue discharge headways accepted by queued drivers at red signals? The second is the establishment of a clear analysis method needs to indicate if signalized approaches on roundabouts would perform better in terms of capacity when compared with unsignalized roundabout approaches. Most of the existing literature on signalized roundabouts are focused on reporting observations of a field implementation, and do not feature any sort of design of experiments that would yield information on when and how to appropriately signalize a roundabout.

2.3.5 Costs Associated with Roundabout Signalization

The signalization of a roundabout facility has an unintended consequence of adding cost for the hardware, electricity, and maintenance of signal equipment at the site, and it must be clear that the benefits of implementing the signalization will outweigh the costs generated. On the cost side of the equation, the Washington State DOT estimates the cost for purchasing, installing a traffic signal would range from $250,000 to $500,000 in addition to a maintenance cost of approximately $8,000 annually [46].

A primary benefit of signalizing roundabouts is the reduction in average delay per vehicle at the facility, which can theoretically be monetized in terms of the cost of travel time for users and the anticipated travel time savings. The U.S. Department of Transportation (USDOT) releases a document titled the Departmental Guidance for

Conducting Economic Evaluations Report [47], which estimates the current value of travel time (VTTS) to be $14.10 per hour per person. In order to perform the necessary calculations, the anticipated total travel time savings per day would need to be calculated (not just peak-hour operations) and a yearly rate computed in order to predict out the number of years until the return on investment was reached. This calculation would be complicated by the ongoing costs in electricity and maintenance of the equipment. A further layer of complication comes in from the anticipated theoretical safety benefits of a roundabout over a signalized intersection, and the monetization of crashes compared with the reduction in severity and rate of crashes at the site.

However, all of these calculations taken into account, a more realistic outlook on the cost aspect of deciding to implement a signalized roundabout would be to look at the comparative costs between signalizing an existing roundabout compared with reconstructing the facility as a conventional signalized intersection, or at the costs of constructing a new signalized roundabout facility compared with constructing a new conventional signalized facility. Each location will have different constraints and different existing conditions to be worked with, and cost assessments will need to be done on a case-by-case basis.

CHAPTER 3 ASSESSING ROBUSTNESS OF PLANNING TOOLS FOR INDICATING ROUNDABOUT PERFORMANCE

Abstract

Delay is considered as the primary measure of effectiveness for both signalized and un-signalized intersections according to the Highway Capacity Manual (HCM). However, simpler methodologies are often more practical for purposes of preliminary design engineering. This research seeks to determine if a simplified methodology, the critical sum method, can serve as an indicator variable for the HCM average control delay of a single-lane roundabout.

With a simple mental calculation, the critical sum method can provide an indication of performance of a signalized or unsignalized intersection or interchange, but the reliability of this prediction when translated to a delay measure is not well-documented. Using the algorithmic formulations for both the critical sum method and the HCM method, 250,000 volume scenarios were run, systematically varying parameter combinations with a full-factorial experimental design. Although the critical sum value was found to provide accurate predictions about the average vehicle delay for low-volume conditions, up to 15 seconds of delay per vehicle, ultimately the CSM for higher-volume conditions was found to be a poor predictor of average delay. Volume scenario factors such as directional split and turn movement percentages had minimal effect on these findings.

3.1. Background

In the 1960's, the United Kingdom small-radius traffic circle that increased safety and improved traffic operations by effecting yield control on vehicles entering the facility and designing the horizontal alignment to reduce circulating speeds. Today this geometric configuration, widely known as a modern roundabout, is common place and a standard solution for engineers both within the United States and abroad. The modern roundabout has several advantages over conventional signalized intersections, most notably a drastic reduction in conflict points, and reduced speed that minimizes severity of crashes that do occur [1]–[6].

The primary operational performance metric for a junction is the average delay per vehicle, with secondary parameters including safety, queuing, cost, emissions, etc. When calculating a facility's Level of Service (LOS), the Highway Capacity Manual [48] utilizes average delay per vehicle as the primary measure of effectiveness for both signalized and unsignalized intersections. The chart listing the various thresholds of delay for each level of service is shown below in **Figure 3-1**.

Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio[a] $v/c \leq 1.0$	$v/c > 1.0$
0–10	A	F
>10–15	B	F
>15–25	C	F
>25–35	D	F
>35–50	E	F
>50	F	F

Note: [a] For approaches and intersectionwide assessment, LOS is defined solely by control delay.

Figure 3-1 LOS criteria for roundabouts in 6th Edition of HCM [48]

Moreover, only the control delay is considered in the HCM methodology, with control delay defined as the time a driver is involved in various queuing situations, such as deceleration upon approach, entering, exiting, and wait time for an acceptable gap at the front of the queue when entering circulating flow [7]. It notably does not include additional travel time due to geometric design of the facility, which makes it less useful of a measure when comparing alternative intersection designs, or an interchange against an at-grade alternative. For a single-lane roundabout, the HCM method calculates the average control delay using a logarithmic mathematical equation, utilizing lane capacity, volume to capacity ratio, and analysis period as the input parameters.

Current practice for roundabout operations focuses on identifying individual approach capacity as a function of circulating and entering flow, based on flow definitions previously shown in **Figure 2-6**, along with a sample calculation of circulating flow shown in **Figure 2-7**. While geometric elements can impact the overall flowrate of an approach, the circulating flow is the primary constraint. The circulating flow directly impacts the availability of adequate gaps in the roundabout, with low flow equating to more gaps and wider gaps, allowing drivers to enter without significant delay [8].

For the entry point on modern roundabout approaches, gap acceptance is modeled as being directly influenced by two parameters, the critical headway and the follow-up headway. Critical headway is measured on the circulating lane, and is the minimum time between two successive circulating vehicles required by a driver to safely enter the roundabout between those vehicles. The National Cooperative Highway Research Program (NCHRP) Report 572 defines critical headway as greater than the largest headway rejected and shorter than the accepted headway [11]. Critical headway has also been found to be influenced by local conditions such as: driver behavior, geometric layout, and prevailing traffic conditions [13]. Follow-up headway is measured on the approach lane, and is defined as the minimum time between two successive vehicles entering a roundabout into the same circulating gap. The follow-up headway is estimated by taking the average difference between the passage times of two entering vehicles accepting a gap in the conflicting stream under a queued condition [7].

There are several methodologies for calculating the volume-to-capacity ratio of roundabout approaches. The critical sum method uses the most basic calculations of the methods discussed here, providing a measure of demand for a given approach by adding the volume of the entering flow (v_e) to the volume of the circulating flow (v_c) opposing it. The capacity against which this demand is compared against is taken by default as equal to 1,600 veh/h/ln independent of other factors, which represents the total number of vehicles in conflict with each other that can be serviced by a facility [21]. This value of 1,600 is a constant used for all intersection geometries, with minimal literature examining this assumption [22]. In contrast, the current 6th edition of the Highway Capacity Manual (HCM) calculates the maximum capacity of a given approach using an exponential regression model based on gap acceptance, determining the volume-to-capacity ratio based on the demand flow rate of the approach [7].

3.2. Objective & Purpose

The main scope of this chapter is to explore the degree to which the critical sum method can provide a planning-level assessment of roundabout performance that will be consistent with the results of more formal analysis using HCM delay procedures. In assessing the effectiveness of the critical sum method as a planning tool for preliminary engineering, two primary objectives are identified. The first is to assess the reliability of the CSM value as an indicator of HCM delay; does knowing the value of CSM provide valuable information about what the value of HCM delay is likely to be? The second is to assess if the ability of the CSM value to predict delay is impacted by the type of volumes experienced in a given situation; for example, if the flows are balanced do the predictions get better?

3.3. Evaluation Approach

As a measure of reliability, the percentage of the data points within ±5 seconds of the observed mean delay will be identified. This will define the maximum range within which the critical sum method is recommended to be used by practitioners for providing valid and effective results, as knowing the delay accurately within five seconds of an estimate will give confidence regarding which level of service a facility can be anticipated to operate.

A systematic approach was developed to generate sets of origin-destination volumes that include a broad spectrum of demand flowrate combinations. Next, the algorithmic formulations for the critical sum method and the HCM average delay per vehicle values were applied to each volume scenario, based on the 6th edition formulation [48]. Finally, the reliability of the critical sum method as an indicator variable of HCM average delay per vehicle at a single-lane roundabout was assessed.

3.3.1 Origin-Destination Volume Combinations

Volume scenarios were generated systematically and with randomization built in, as shown in **Table 3-1** in order to try and thoroughly cover the breadth of volume combinations encountered in practice. Major and minor approach two-way volumes, splits, and turn-movement percentages are varied. The major and minor approach two-way volumes range from 50 veh/h to 2000 veh/h. The split factor ranges from 0.5 to 0.7. Similarly, the turning-movement factor ranges from 0.05 to 0.25. Ultimately, the permutation of these analysis parameters results in 250,000 scenario combinations. Randomization is introduced to the individual volume scenarios, but the degree of randomization is always set to be less than the step size used. As an example of the randomization used, there are 12,500 volume scenarios that begin with an east-west total volume of 100 pce/h, but with randomization there will be 12,500 different values for east-west total volume that are uniformly distributed between 50 and 150. The reason for this randomization is to generate as wide a variety of volume scenarios as possible within the confines of the study, since there are a great number of different volume combinations that result in the same critical sum value.

Table 3-1 Volume scenario parameter permutations and perturbations

Direction	Parameter	Units	Min	Max	Step	Values	Randomization
EW	2 Way Volume	pceph	100	2,000	100	20	-50+[100*Rand(0,1)]
	Split percent	%	0.5	0.7	0.05	5	-0.025+[0.05*Rand(0,1)]
	Turn percent	%	0.05	0.25	0.05	5	-0.025+[0.05*Rand(0,1)]
NS	2 Way Volume	pceph	100	2,000	100	20	-50+[100*Rand(0,1)]
	Split percent	%	0.5	0.7	0.05	5	-0.025+[0.05*Rand(0,1)]
	Turn percent	%	0.05	0.25	0.05	5	-0.025+[0.05*Rand(0,1)]
Total	250,000 volume scenarios						

For a given approach, combinations of two-way volume and directional split lead to entry demand volumes that range from 30 veh/hr up to 1,400 veh/hr. For comparison, the calibrated equation for capacity at roundabouts (as shown in Figure 2-9) for one lane entering against one lane circulating ranges between 1,380 veh/hr able to enter when 0 veh/hr are circulating, to 270 veh/hr able to enter when 1,600 veh/hr are circulating [11]. As such, even the extreme analysis events within our dataset with 30 veh/hr entering against 1,400 circulating and 1,400 entering against 30 veh/hr circulating is within the range of what is commonly analyzed by the HCM methodology.

3.3.2 Analysis Methodology & Equation Documentation

The critical sum method estimates intersection demand by summing the combination of the critical movements at any intersection or interchange facility, signalized or unsignalized [21]. The resulting value represents the demand volume attempting to pass through an intersection during an hour, in units of vehicles per hour per lane (veh/hr/ln). The main benefit of the critical sum method for use as a preliminary design tool is the simplicity of its calculation, without need for knowledge of signal timings at signalized intersections. Calculations of the critical sum method at roundabouts result in a different value at each approach, so two measures were investigated for this research, the maximum critical sum experienced by the worst-case approach (CS_{MAX}), and the weighted average of all approaches ($CS_{weighted}$). The value of critical sum on each approach (CS_{i}) is found based on the calculation shown in *equation 3-1*, and sums the conflicting circulating flow ($\mathrm{v}_{\mathrm{c,i}}$) with the entering flow ($\mathrm{v}_{\mathrm{e,i}}$).

$$CS_i = v_{e,i} + v_{c,i} \qquad \textit{Eq. 3-1}$$

$$for\ i =$$

$$eastbound, northbound, westbound, and\ southbound\ approaches$$

$v_{e,i}$ = the demand flowrate entering on approach *i* (veh/hr)

$v_{c,i}$ = the circulating flowrate passing in front of approach *i* (veh/hr)

The maximum critical sum (CS_{MAX}) is found by selecting the highest value between the four approaches as the governing capacity for the roundabout as shown in *equation 3-2*. The weighted critical sum ($CS_{weighted}$) considers the weighted average of the four approaches as shown in *equation 3-3*.

$$CS_{MAX} = MAX[CS_i] \qquad \textit{Eq. 3-2}$$

$$for\ i = eastbound, northbound, westbound, southbound$$

$$CS_{weighted} = \frac{\sum_i (CS_i * volume_i)}{\sum_i volume_i} \qquad \textit{Eq. 3-3}$$

$$for\ i = eastbound, northbound, westbound, southbound$$

The HCM methodology for calculating the capacity of an approach (c_i) on a single-lane roundabout is shown in *equation 3-4*, and the resulting delay on that approach (d_i) in *equation 3-5*.

$$c_i = 1380 * exp^{(-0.00102 * v_{c,i})} \qquad \textit{Eq. 3-4}$$

$$d_i = \frac{3600}{c_i} + 900T\left[x_i - 1 + \sqrt{(x_i - 1)^2 + \frac{\left(\frac{3600}{c_i}\right)x_i}{450T}}\right] + 5 * min[x_i, 1] \qquad \textit{Eq. 3-5}$$

$for\ i =$

$eastbound, northbound, westbound, and\ southbound\ approaches$

d_i = average control delay for approach *i* (seconds/vehicle)

$x_i = \frac{v_{e,i}}{c_i}$ = volume-to-capacity ratio of approach *i* (unitless)

c_i = capacity of approach *i* (vehicles/hour)

T = analysis time period (hour)

An example analysis for a given volume scenario is presented in **Table 3-2**, below.

Table 3-2 Sample volume scenario analysis

Input Parameters	T = 60 min				Volume Application					Critial Sum Method			HCM 6th Method			
Approach		**road vol**	**dir split**	**turn %**	**app vol**	**L**	**T**	**R**	**circ vol**	**CS**	**CS (max)**	**CS (weight)**	**cap.**	**v/c ratio**	**app delay**	**int. delay**
1	**EB**	800	0.6	0.1	480	48	384	48	359	839	839	758	957	0.50	10.0	8.58
3	**WB**				320	32	256	32	316	636			1000	0.32	6.9	
2	**SB**	700	0.55	0.15	385	58	269	58	335	720			981	0.39	8.0	
4	**NB**				315	47	221	47	490	805			837	0.38	8.8	

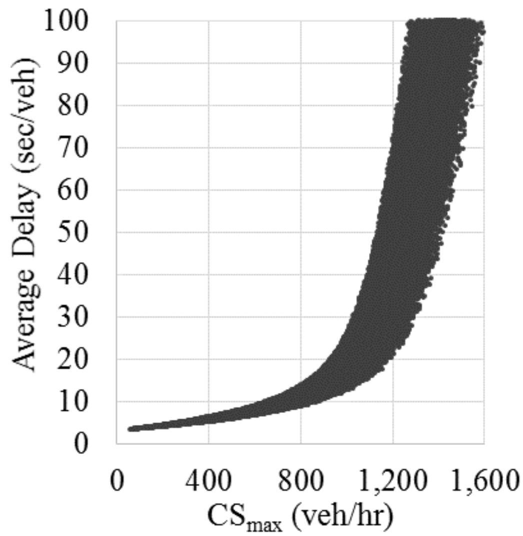
3.3.3 Assessing Reliability of CSM as an Indicator Variable for Average Delay

The ultimate goal is to assess the degree to which the value of the CSM provides useful information about the anticipated value of HCM average delay per vehicle for a given volume scenario on a roundabout facility. For a given value of CSM, there exists a “most likely" expected value of HCM average delay per vehicle, and there also exists the variety of individual results generated by the volume combinations assessed herein, which may or may not be distributed close to the mean result.

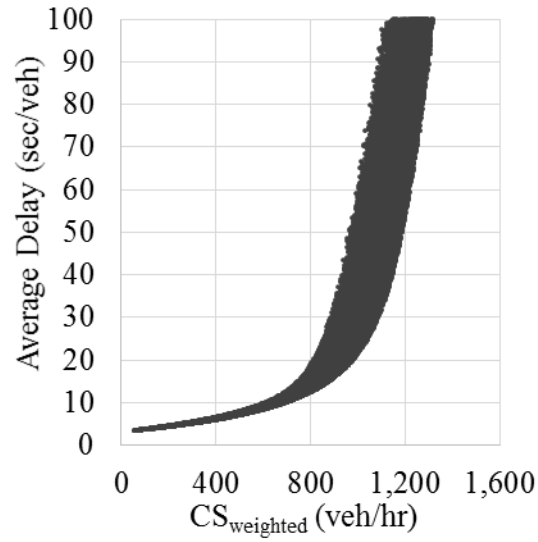
To establish an expected value for each value of CSM, bins were defined in groups of 100 veh/hr/ln; for example, all volume scenarios resulting in critical sum values between 550 and 650 were binned into the 600 grouping. The mean of the HCM delay results for each CSM bin was determined. The previously defined criteria of ±5 seconds of the mean observed value was then assessed, with the goal of having 95% of all observations within a given bin falling within that ±5 seconds range.

3.4. Results of Algorithmic Calculations

Ideally, for the critical sum measure to be a perfect indicator variable, all volume combinations resulting in a given critical sum value would also result in a constant value for average delay. The question that follows is whether the individual results deviate from the expected value within an allowable range. The volume scenarios investigated ranged from very-low volumes up to combinations well in excess of capacity for the one-lane roundabout facility. In all, 250,000 volume scenarios were run, calculating values for the maximum critical sum (CS_{MAX}), the weighted average of the critical sum ($CS_{weighted}$), and the HCM delay. While the dataset includes far more volume combinations than are likely to be encountered in the field, the author zeroed in on a range of interest included critical sum values up to 1,600 and delay values up to 100 seconds/vehicle. While many of the points overlap and occlude information about the density of points, the range of results is shown in **Figure 3-2**, showing delay plotted against maximum critical sum (CS_{MAX}) on the left **(a)**, and against weighted critical sum ($CS_{weighted}$) on the right **(b)**.



(a) Maximum approach critical sum



(b) Weighted critical sum

Figure 3-2 Critical sum (veh/h) versus HCM average control delay (sec/veh)

The initial results shown in **Figure 3-2** visually indicate a small amount of deviation in average delay values for volume scenarios resulting in CSM values up to around 800 veh/hr/ln, with an increasing deviation (vertical width) as the CSM values approach and exceed capacity. Using the weighted average of the critical sum value visually shifts the plot toward the left on the chart, but doesn't appear to provide a significant improvement in terms of reducing the vertical width of data points as the average delay per vehicle climbs above around 10 seconds. Because the weighted average does not provide practical gains to reliability for the reliability of CSM as an indicator variable, further investigation will focus on the maximum critical sum value (CS_{MAX}), which is consistent with its application in software applications such as CAP-X.

3.5. Analysis of Results

Further analysis of the results focuses on the distribution of HCM delay values corresponding to a given value of CSM.

3.5.1 Maximum Critical Sum versus HCM Delay

For each set of volume scenarios resulting in a given critical sum value, a mean delay was identified as shown in **Figure 3-3** and **Table 3-3**. Additional information in **Figure 3-3** includes the range of the individual results for delay for each volume scenario resulting in a given critical sum.

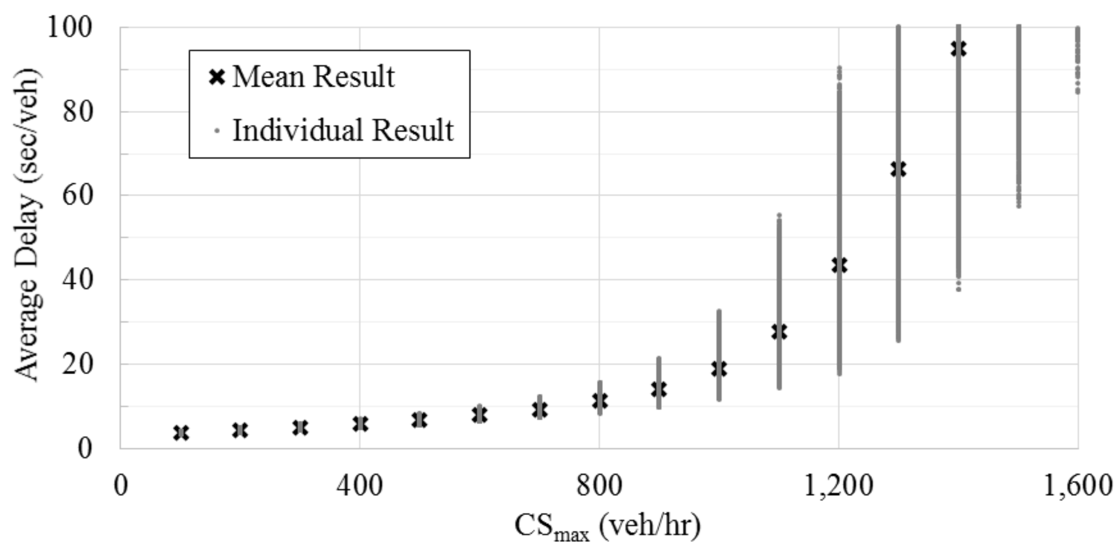
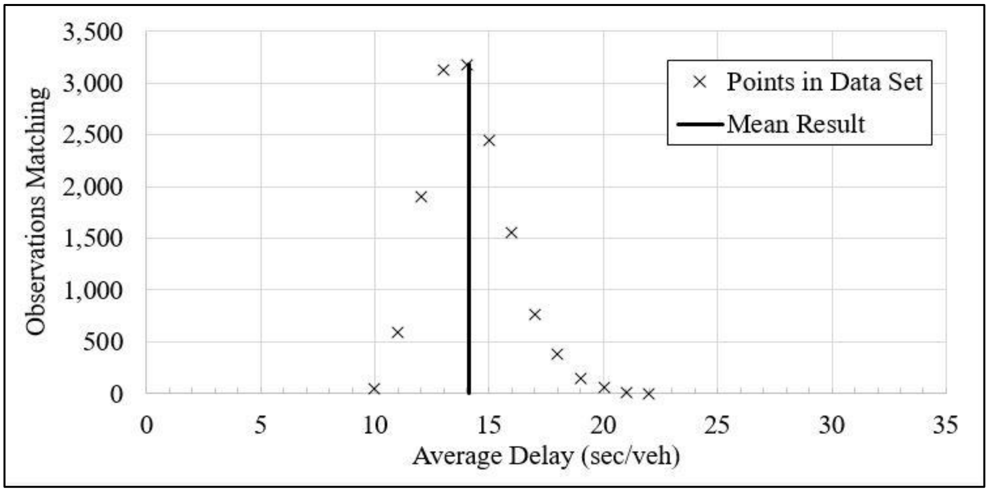


Figure 3-3 Maximum approach critical sum value versus HCM average delay per vehicle

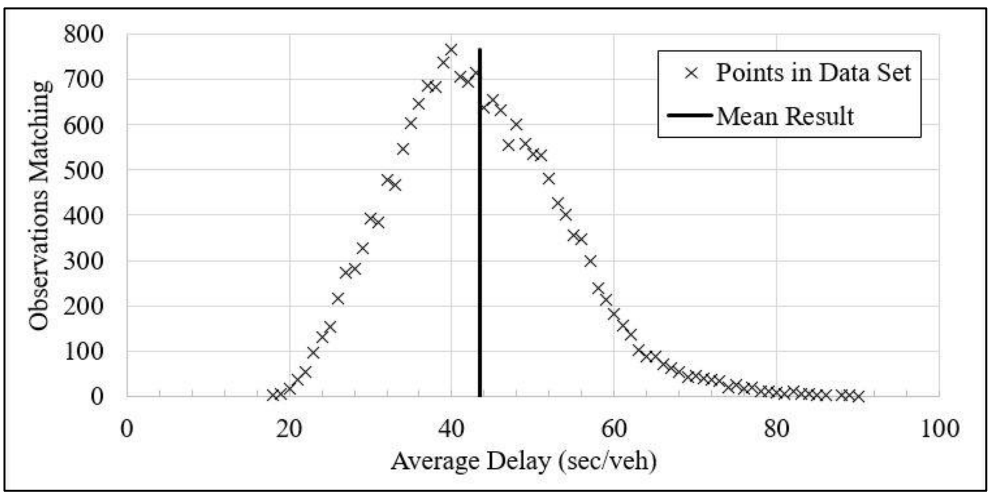
Table 3-3 Maximum approach critical sum value versus HCM average delay per vehicle

Critical Sum (Max)	Mean Delay	Standard Deviation	Count of Data	Count of Data Mean ± 5 sec.	Percent of Data Mean ± 5 sec.
100	3.8	0.1	710	710	100%
200	4.3	0.2	2,389	2,389	100%
300	5.0	0.3	4,090	4,090	100%
400	5.8	0.3	5,742	5,742	100%
500	6.7	0.4	7,456	7,456	100%
600	7.9	0.6	9,108	9,108	100%
700	9.3	0.8	10,759	10,759	100%
800	11.3	1.1	12,456	12,456	100%
900	14.1	1.8	14,195	14,074	99%
1000	18.9	3.1	15,834	14,333	91%
1100	27.8	6.1	17,506	10,185	58%
1200	43.4	10.6	18,870	6,695	35%
1300	66.4	15.0	19,540	5,307	27%
1400	95.0	18.8	19,329	4,461	23%
1500	129.2	23.8	18,095	3,360	19%
1600	169.6	30.9	16,172	2,257	14%
1700	217.3	40.8	13,799	1,199	9%
1800	271.0	50.7	11,793	850	7%
1900	332.1	61.2	9,621	554	6%
2000	395.2	69.0	7,750	411	5%

Referring back to the results shown on **Table 3-3**, the CSM (CS_{MAX}) was found to be an accurate indicator variable for HCM delay up to a value of 900 veh/hr/ln, corresponding to an HCM average delay per vehicle of 14.1 seconds. However, volume scenarios resulting in CSM values of 1,000 and higher experience increasingly smaller percentages of delay values within the ±5 second range. Examining a vertical slice through the data, delay values are binned to the nearest second, and a plot of the number of results sharing a common critical sum and delay value is shown in **Figure 3-4**, including example slices along critical sum values of 900 **(a)** and 1,200 **(b)**.



(a) Sub-set of data with critical sum equal to 900 ± 50



(b) Sub-set of data with critical sum data equal to 1,200 ± 50

Figure 3-4 Distribution of average delay results relative to a given critical sum value

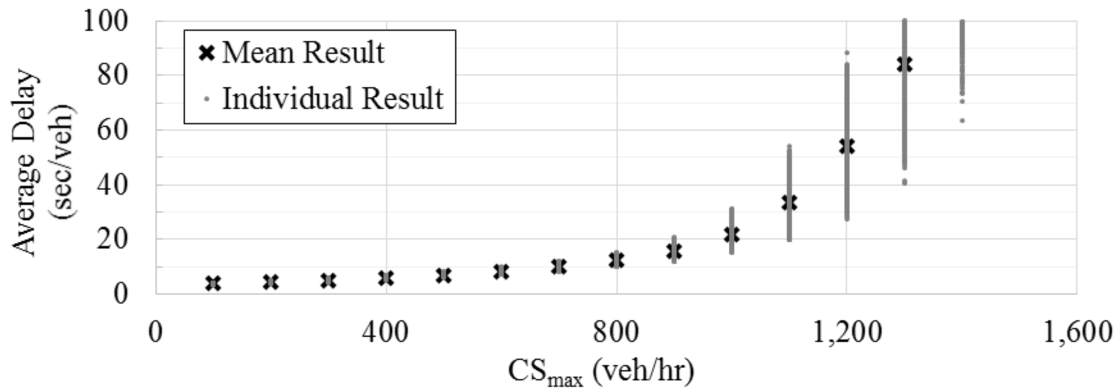
As seen in **Figure 3-4**, the mean value of HCM delay for all volume scenarios resulting in a critical sum value near 900 veh/hr/ln is near to 14 seconds, with the majority of the data falling between 10 and 20 seconds. In contrast, the volume combinations resulting in a critical sum value of 1,200 veh/hr/ln result in a mean HCM delay measure of around 42 seconds per vehicle, but the individual results range between

20 and 80 seconds. Knowing that the value of the CSM value is 900 gives a great deal of information regarding the expected value of the HCM delay, allowing an engineer to confidently predict that the level of service for the facility will be a B or a C. However, knowing that the CSM value is 1,200 gives considerably less information about the anticipated value of the HCM delay, and an engineer could predict that the level of service of the facility will be somewhere between C and F.

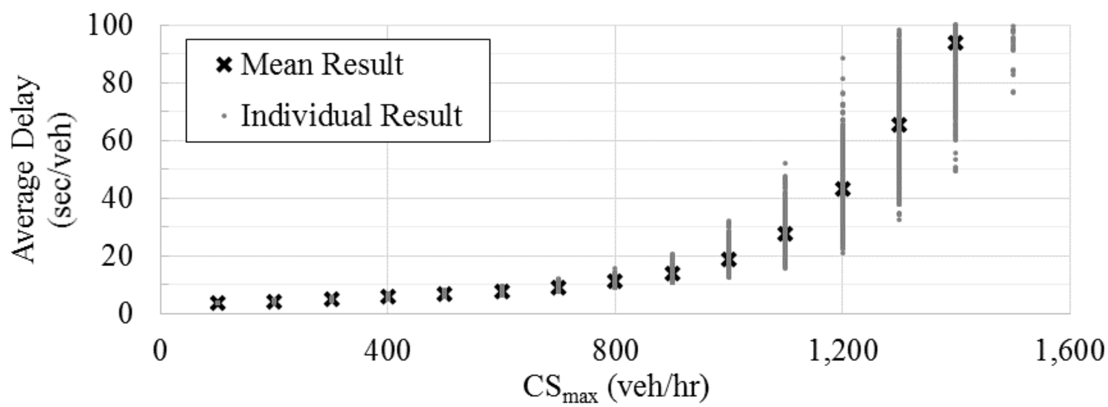
3.5.2 Impact of Volume Scenario on the Reliability of Critical Sum in Predicting Delay

Although the results indicate that the CSM is an unreliable indicator of HCM delay results at higher volumes, it is possible that the nature of the traffic scenario would impact this finding. To this end, sensitivity analysis was conducted on both the directional split and turn percentage factors to determine if their values impacted reliability.

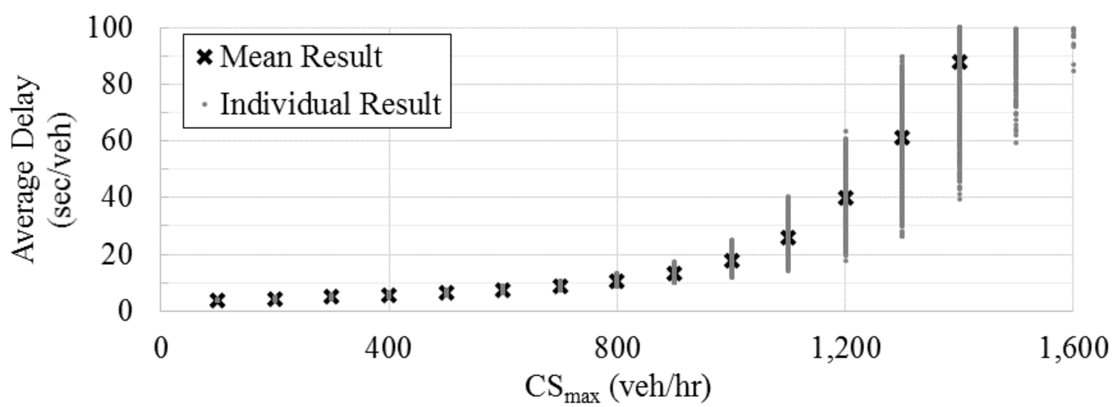
The impact of directional split on the relationship between critical sum and delay is shown below in **Figure 3-5**. Three different sub-sets of the data were analyzed, including: **(a)** both roadways having balanced flow with a directional split of 0.5; **(b)** one roadway having balanced flow with a split of 0.5, and one roadway having unbalanced flow with a split of 0.7; and **(c)** both roadways having unbalanced flow with a split of 0.7.



(a) Sub-set of scenarios with directional split equal to 0.5 for both roadways



(b) Sub-set of scenarios with directional split of 0.5 for one roadway and 0.7 for the other

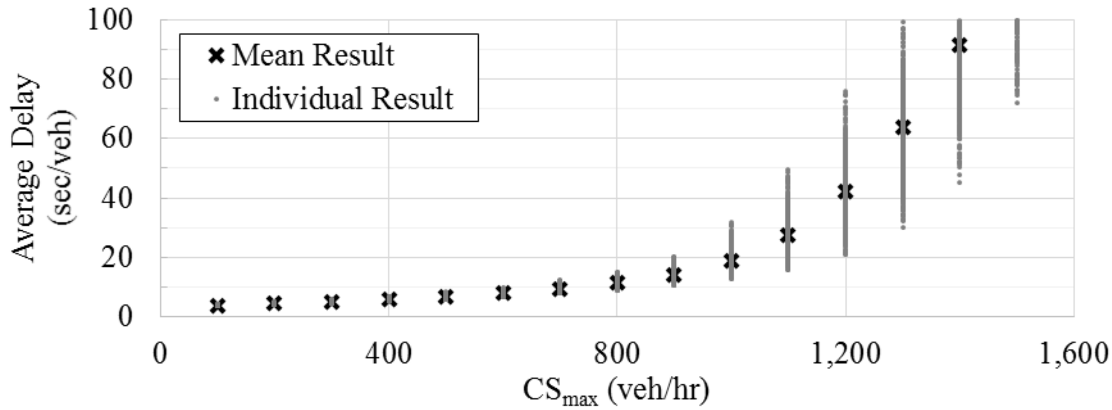


(c) Sub-set of scenarios with directional split equal to 0.7 for both roadways

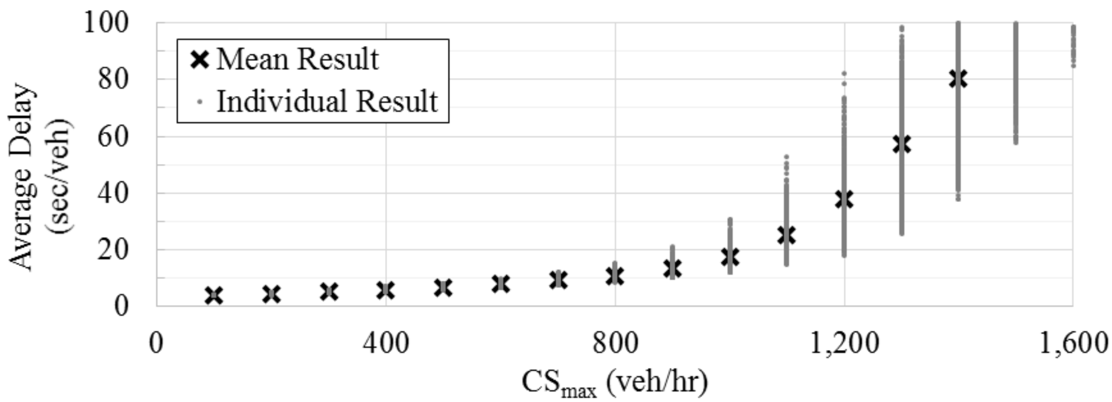
Figure 3-5 Impact of directional split on relationship of critical sum and delay

The findings do not support the idea that certain combinations of directional split have either a positive or negative impact on the usefulness of CSM as a predictor of HCM delay.

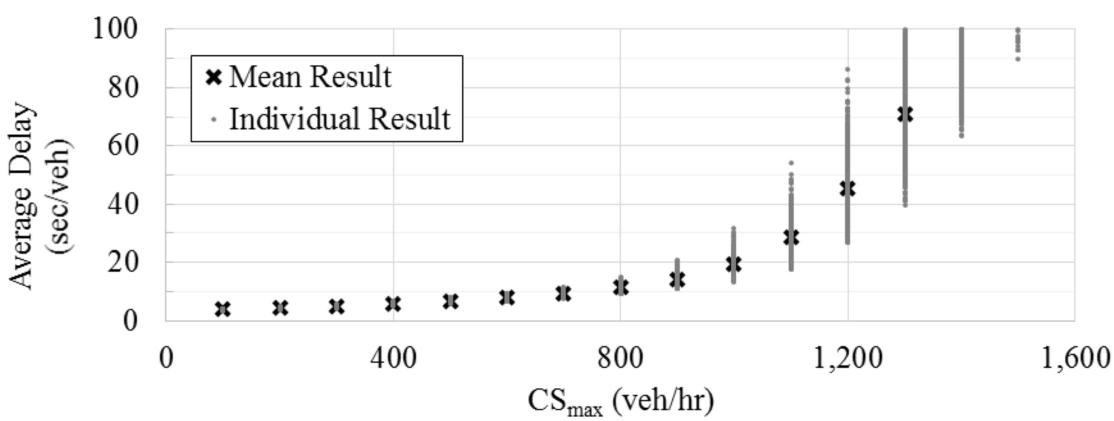
The impact of turn percentage on the relationship between critical sum and delay is shown below in **Figure 3-6**. Three different sub-sets of data were analyzed, including: **(a)** both roadways having low turn-movement percentages of 5%; **(b)** one roadway having a low turn-movement percentage of 5%, and one roadway having a high turn-movement percentage of 25%; and **(c)** both roadways having high turn-movement percentages of 25%.



(a) Sub-set of scenarios with turn percentage equal to 5% for both roadways



(b) Sub-set of scenarios with turn percentage of 5% for one roadway and 025% for the other



(c) Sub-set of scenarios with turn percentage equal to 25% for both roadways

Figure 3-6 Impact of turn percentage on relationship of critical sum and delay

The findings do not support the idea that certain combinations of turn percentages have either a positive or negative impact on the usefulness of CSM as a predictor of HCM delay.

3.6. Research Limitations

This study relied heavily on the generation of a large number of volume combinations to thoroughly examine whether CSM is a practical indicator variable for HCM delay. However, not all volume scenarios are equally likely to occur in the field, and it is possible that the outlier values are less likely to be observed and the values closely distributed near the mean result for HCM delay are actually much more likely to exist in the field. In the absence of field data all volume scenarios were provided an equal weighting in terms of importance, and additional data may have resulted in a different set of conclusions.

3.7. Conclusions and Recommendations

This research focused on the practicality of using critical sum method, a capacity based approach, for anticipating the HCM average delay per vehicle at a single-lane roundabout facility. Experiment design included the generation of 250,000 volume scenario combinations, with an examination of the distribution of HCM delay values within a set of volume scenarios that resulted in a common value for CSM.

The critical sum data points were plotted against the delay, with the variability of the delay results increasing as the critical sum values increased. The highest value of the critical sum method that was determined to be reliable was 900 veh/ln/hr, which corresponded to an HCM delay value of 14.1 seconds. Volume scenarios resulting in higher

CSM values had too large of a range of delay values to use the CSM as a practical predictor variable.

Sensitivity analysis was conducted on both the directional split and turn percentage factors to determine if their values impacted these findings, but the results show that the impracticality of the CSM method for predicting values of delay in excess of LOS B is independent from the type of volume scenario analyzed.

Using the critical sum method was found to confidently anticipate whether the Level of Service (LOS) for a roundabout is an A or a B, but was not found to provide reliable results when they are most needed, attempting to determine if the facility will operate at or near failure in the LOS C, D, E, or F range.

The benefit of not needing signal timings for analyzing conventional signalized intersections will ensure that the CSM method continues to be used widely in engineering practice as a preliminary engineering tool. It is the recommendation based on these research findings that any application using capacity-based measures for signalized intersections should instead use the full HCM methodology to calculate roundabout operations, a relatively simple exercise as the inputs are limited to circulating volume, entering volume, and analysis period.

CHAPTER 4 SIGNALIZED METERING OF ROUNDABOUTS TO BALANCE CAPACITY ON APPROACHES

Abstract

This chapter explores the potential impact of applying metering signals on roundabouts. The volume conditions and signal timing leading to a successful implementation of a metered roundabout are explored with two investigations. The first highlights the theoretical concept of creating artificial gaps with a metering signal, loading the simulation model with only traffic on two approaches of the roundabout. The second loads traffic on all four approaches for a more realistic test of the signal metering concept. Traffic micro-simulation software VISSIM is used for both investigations, calibrating gap acceptance and follow-up headway to generate delay results consistent with the calculated delay from the Highway Capacity Manual, 6th Edition. Although the results of the first investigation yield positive results, the theoretical benefits of the metered roundabout were not validated by the second investigation using a full volume scenario. It is possible that the driver behavior parameters calibrated to match the operational results of the HCM may be confounding the benefit created by the metering signal, and further study is recommended.

4.1. Background

In recent years, the roundabout has increased in popularity in the United States as a safe and efficient traffic control method for junctions. However, the capacity of the roundabout is limited relative to a signalized intersection, and it is more susceptible to a reduction in performance due to one approach having higher volumes than others, as it lacks the ability of a signalized intersection to redistribute green time as needed. During commuting traffic in the a.m. and p.m. peak periods, surface roadway networks routinely encounter directional flow patterns caused by prevailing traffic that moves from home to work in the morning and from work to home in the afternoon. A location that might otherwise benefit from the safety and off-peak performance benefits of a roundabout over a signalized intersection may have to reject the roundabout solution due to this common type of peak-hour flow pattern.

Average delay of vehicles on each approach of a roundabout is directly related to the demand and capacity of that movement, with capacity in turn being based on the number of vehicles circulating in front of the entering approach lane(s). The higher the number of vehicles passing in front of an entry-point to the roundabout, the fewer the openings of adequate size there will be in that circulating traffic for vehicles to pull into the facility.

4.1.1 Roundabout Signalization

Signalization of one or more approaches on the roundabout could create additional gaps in the circulating traffic, redistributing the capacity of the facility. In a few isolated cases where a roundabout facility experienced higher than anticipated traffic growth, signalization has been used as a temporary solution, a stop-gap measure before a more permanent solution can be completed, such as reconstruction of the facility as a conventional signalized intersection [35], [44], [49].

The earliest example of roundabout signalization found in the literature was in the United Kingdom on large roundabouts (rotaries) for controlling traffic movement. The County Surveyors' Society canvassed 49 road authorities to identify the reasons for signalization of roundabouts, with 42% identifying queue control, 39% increasing capacity, and 17% reducing the accident rate [38]. A number of studies also identified these justifications when applying signalization to roundabout facilities [37]–[39], [42]–[44], [49].

Hallworth [39] and Stevens [43] both discuss the various ways in which signal control has been applied on roundabouts, with Natalizio developing something of a taxonomy, as shown previously in **Table 2-2**. Hallworth's taxonomy of signalized control for roundabout facilities includes the means of control, whether the signal is operated part time or full time, and whether full control is implemented (on every approach), or if only some approaches are signalized. A facility experiencing direct control will have signalized control applied at the entry points to the circulating lane, while a facility utilizing indirect control will use upstream signalization, either mid-block or at an adjacent signalized intersection, to attempt to control the demand flowrate for one or more approaches. A signalized roundabout with part-time signalization may use time-of-day programming to determine when the signal is used, or may implement actuated control with detectors determining when the criteria are being met to justify use of the signal. Finally, full-signalization is often implemented as a partial fix before a facility is reconstructed as a conventional signalized intersection, while partial-signalization controls only some approaches at the facility, and has been used as a longer-term solution.

However, most roundabout signalization is commonly implemented to operate on a part-time basis depending on peak hours and heavy demand conditions. With the aid of a case study by Mickleham and Broadmeadows, Akcelik presented an analysis comparing the operating performance of a meter-controlled two-lane roundabout and a conventional unsignalized facility [44]. The results indicate that the signals were successful at transferring the delay and queue from the controlling approach to the controlled approach.

A summary of the operation, control, and timing of roundabout signalization was provided by Akcelik [50]. An elementary concept of an analytical model was reported, and the duration of the red and blank signal was highlighted. Natalizio worked to verify the robustness of Akcelik's conceptual model by coding the model in an Excel spread sheet [38], [51]. Additionally, Natalizio provided guidance for when a roundabout requires a signal, and what signal timings were helpful. Higher signal cycle times were found to benefit the controlling approach, with lower signal cycle times benefits the metered approach. Natalizio further found that the required demand flowrates for metering a single lane roundabout is when the combination of entry and circulating volumes is between the range of 1300 and 1400 vehicles per hour, with performance beginning to decline once demand increases to the range between 1550 and 1650 vehicles per hour. Another study performed a quantitative comparison between a signalized and an unsignalized roundabout under critical traffic conditions, indicating that unsignalized roundabouts perform better at low traffic volumes due to the negligible queue delay, whereas signalized roundabouts perform better at high traffic, since signals produce a more balanced delay leading to an overall improvement in capacity [40].

4.1.2 Existing Implementations

Within the English-language literature, examples of roundabout signalization have been found in Australia, the United Kingdom, and the United States of America. The primary purpose of these implementations has been to avoid excessive queuing and delays at the approaches affected by high directional flows [39], [43]. A study done by Stevens briefly reported some of implemented signalized roundabouts as presented in **Table 4-1** [43].

Table 4-1 Examples of signalization of roundabouts [43]

Roundabout	Means of control	Time of operation	Approach control	Problem areas
Charles Street, MD, USA	Indirect	Full	All	Unbalanced flow
Clearwater, FL, USA	Indirect	Full	One	Unbalanced flow during spring break
Penn Inn, Abbot, UK	NA	Full	All	Unbalanced flow and excessive turning movements

Unfortunately, the only two documented cases of roundabout signalization within the United States are one case in Maryland and one in Florida. The site in Clearwater, Florida [42] features a signal actuated by a queue detector, installed on the highest-volume approach to create gaps for downstream traffic. At this location, the high-flow approach consistently prevented entrance to the facility by the downstream approaches, causing their queues to back up into adjacent intersections during the peak period of the day. At the

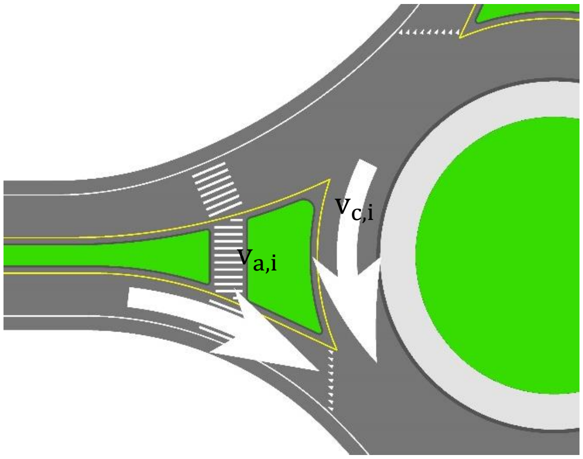
Charles Street facility in Maryland, a traffic signal was installed because two legs of the roundabout represented over 90% of the flow, and it was deemed necessary to prioritize capacity for the higher-volume approaches over the lower-volume ones.

4.2. Research Objectives

This research seeks to identify the potential of signalization on single-lane roundabouts to redistribute capacity from a low-demand approach to an adjacent high-demand approach. Two main objectives have been identified to analyze the usefulness of signalization at roundabout facilities. The first is identifying the volume conditions under which installation of a signal be effective for improving the highest-volume approach as well as the overall facility. The second is optimizing the signal timing, and understanding how this timing should vary based on the demand volumes. The results of this study will be of value both to researchers for further investigation into the use of signalization on roundabouts, as well as to practitioners as a cost-effective improvement for roundabouts failing in performance.

4.3. Methodology

This research seeks to explore the general potential and limits of signalization of roundabouts, and as such a generic roundabout geometry is modeled, shown below in **Figure 4-1**, rather than selecting a field location for analysis. The main dimensions of the roundabout are consistent with the recommendations from NCHRP 672, and include: an inscribed diameter of 150ft, entry width of 20ft, circulating width of 16.5ft, entry radius and exit radius of 75.5ft each, and a lane width of 11.8ft.[8] The geometry is modeled using VISSIM traffic microsimulation software.



$$Demand\ Volume\ in\ Conflict = v_{a,i} + v_{c,i}$$

- $v_{a,i}$ = the demand flowrate on approach *i* (veh/hr)
- $v_{c,i}$ = the circulating flowrate passing in front of approach *i* (veh/hr)

Figure 4-1 Geometric design of roundabout analyzed

4.3.1 Measures of Effectiveness and Simulation Capture of Results

For this research an evaluation parameter of average delay per vehicle, a parameter directly derived from travel time, is selected as the evaluating parameter due to its standardization for use in transportation practice.

Due to the potential for oversaturated conditions leading to growing queue lengths within the simulation, data collection was designed in an attempt to achieve results with minimal bias. The data collection points were placed approximately 1,500 feet away from the centroid of the junction to approximate the distance of an adjacent intersection. Should queue lengths within the simulated environment exceed this distance, the additional wait time incurred by vehicles before they reached this point would be missed, but the result would be consistent with demand starvation by upstream intersections [52]. The length of each simulation run was 25 minutes (1,500 seconds) with no data collected for the first five minutes while the model was populated, and data recorded for vehicles scheduled to enter the model between 5 minutes and 20 minutes, with the final five minutes of simulation time used to ensure that all vehicles were able to complete their path through the facility. Each scenario was run for 20 iterations with varying seed numbers to generate a total of 5 hours of data collection. The purpose behind collecting 15 minutes of data 20 times rather than 5 hours of data once is to avoid bias caused by building queue lengths in the case of near- and over-saturated conditions.

4.3.2 Model Calibration

Taking the HCM delay results as the standard for assessing roundabout performance at the design phase of an engineering study, the gap acceptance and follow-up headway parameters within the VISSIM model were calibrated such that the resulting delay reported by the model matched the predicted results from the HCM model for the same volume scenario.

The calibration was done by conducting a sensitivity analysis on the minimum gap headway for all approaches of a single-lane roundabout in VISSIM to obtain an average delay equal or close to the one attained using the HCM algorithmic equation. A volume of 500 veh/h was assigned on all the approaches of the facility which resulted in having 13.9 seconds as the calculated average control delay per the HCM equation and also yielded 14.2 seconds from the VISSIM simulation run. The calibrated minimum gap headways were 3.1, 2.6, 3.1 and 3.1 seconds for the northbound, eastbound, southbound and westbound approaches respectively. These values were considered as the base values for the calibrated simulation model.

4.3.3 Location of Signalization on Roundabout Facility

This research places the metering signal on the entry point upstream (counterclockwise) of the highest entering volume approach. For example, if the southbound approach has the highest entering volume, the metering signal would be placed at the westbound entry point of the roundabout. Because NCHRP 572 found that non-conflicting vehicles have a negligible impact on the capacity of a roundabout approach, it is not necessary to signalize the other non-critical movements around the facility. This condition is applied only when there is a minimum of ten seconds of difference in the average vehicle delay between the critical and metered approaches, since there must inherently be excess capacity available on the metered approach to transfer to the governing one.

4.3.4 Volume Scenario Methodology

The analysis is divided into two investigations. The first investigation demonstrates the benefits in terms of additional gaps created using a volume scenario that only places vehicles on the controlling and controlled approach, for example, only high volumes on the southbound approach without a signal, and a lower volumes on the westbound approach that is metered with a signal to allow southbound to go. The second investigation looks at directional flow with demand volumes on all four approaches, using equal volumes on three approaches, and a higher volume on the controlling approach.

4.4. First Investigation – Volume on Two Legs

For demonstration purposes of the metering theory, a demand flowrate of 1,000 vehicles/hour is selected for the southbound approach, and a demand flowrate of 700 vehicles/hour is selected for the westbound (metered) approach. The turn percentages are set as 10% lefts, 80% through, and 10% rights for all approaches.

In the case of no metering signal on the westbound approach, the volume scenario selected results in high delays on the southbound approach because there is enough circulating flow in front of the entrance that the southbound vehicles cannot locate a sufficient number of gaps to pull into the circulating lane. In contrast, the westbound approach experiences nominal delay in this scenario, with no conflicting movements circulating in front of its entrance. The addition of a metering signal on the westbound approach introduces an artificial number of gaps per hour to that movement, creating the gaps necessary for the southbound vehicles to share the facility. The first investigation varies the signal timing to examine how the number of artificial gaps introduced per hour impacts the change in delay for each approach.

Two sets of signal timings are used for the first investigation, holding the red time at five seconds and at ten seconds per cycle, and varying the cycle time. As the intention is to create artificial gaps, and the follow-up headway for vehicles entering a roundabout is taken by default as 4.8 seconds/vehicle, a red time on the metered approach of five seconds should introduce a gap for one vehicle to enter during each cycle. Increasing the red time to ten seconds on the metered approach should subsequently provide a gap large enough for two vehicles to enter during each cycle. Seven cycle lengths are tested for the 10-second red option, including 20, 30, 40, 50, 60, 90, and 120 seconds. An eighth cycle length was added for the 5-second red option, additionally including a 10-second cycle. The rate at which artificial gaps are created per hour can be found by dividing an hour's time by the cycle length, such that a 120-second cycle would create 30 gaps per hour, and a 10 second cycle length would create 360 gaps per hour.

4.5. Second Investigation – Volume on all Four Legs

The second investigation includes full volume scenarios with one approach (southbound) having high volume, representing peak-hour directional flow, to further test the artificial gap-creation concept. Seeking to assess volume scenarios in which signalization may be useful, the volume scenarios generated meet the following conditions: the average delay per vehicle for the southbound approach is greater than 30 seconds; and the average delay per vehicle on the southbound approach is at least 10 seconds greater than that experienced by the westbound approach. It is noteworthy that there are some strict upper limits on the applicability of this method; once delay on the southbound approach exceeds 100 seconds per vehicle, signalization of the westbound approach has a marginable impact on improving the overall intersection.

4.6. Results & Analysis

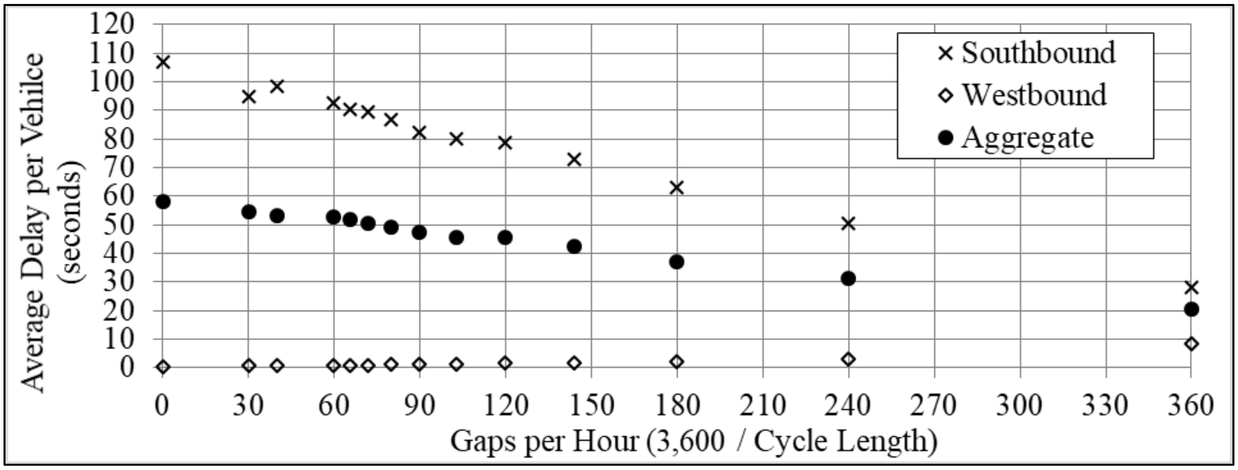
The results of the first investigation are in line with the expectations based on the theoretical approach, but are unfortunately not replicated in the results of the second investigation. The volume scenarios analyzed are shown in **Table 4-2**, below.

Table 4-2 Volume scenarios assessed

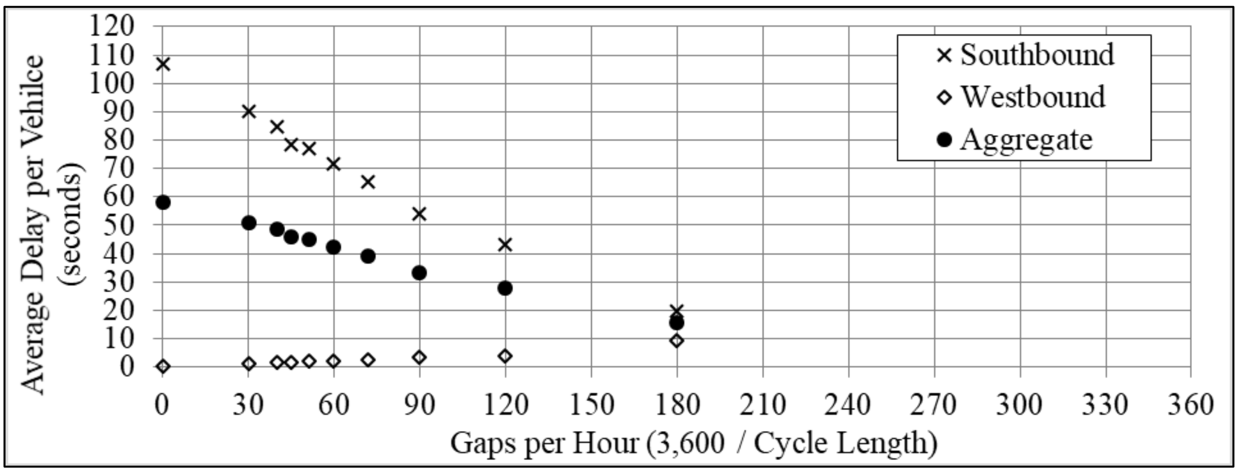
Figure		Red Interval per Cycle (sec)	Approach Volumes			
			Southbound	Westbound	Northbound	Eastbound
2	a	5	1,000	700	0	0
	b	10	1,000	700	0	0
3	a	5	700	500	500	500
	b	5	800	500	500	500
4	a	5	700	600	600	600
	b	5	800	600	600	600
5	a	10	700	600	600	600
	b	10	800	600	600	600

4.6.1 First Investigation Results – Volumes on Two Approaches

For the scenario with volume on two approaches, the shift in delay from the westbound approach to the southbound approach relative to the number of gaps per hour introduced (3,600 seconds divided by the cycle length) is shown in **Figure 4-2**, below. Note, the data point with zero gaps per hour represents the unmetered condition.



(a) 5-second red phase per cycle, 1,000 veh/hr southbound and 700 veh/hr westbound



(b) 10-second red phase per cycle, 1,000 veh/hr southbound and 700 veh/hr westbound

Figure 4-2 Delay implications for signal timing of metered roundabouts – partial volume scenario

The results shown in **Figure 4-2** examine the sensitivity of releasing a southbound approach by installing a metering signal on the westbound approach of a single-lane roundabout, only simulating traffic on those two approaches, with 1,000 veh/hr flowrate southbound and 700 veh/hr flowrate westbound. **Figure (4-2a)** shows the delay results where a 5-second red phase is set for the signal (one vehicle may exit per gap). **Figure (4-**

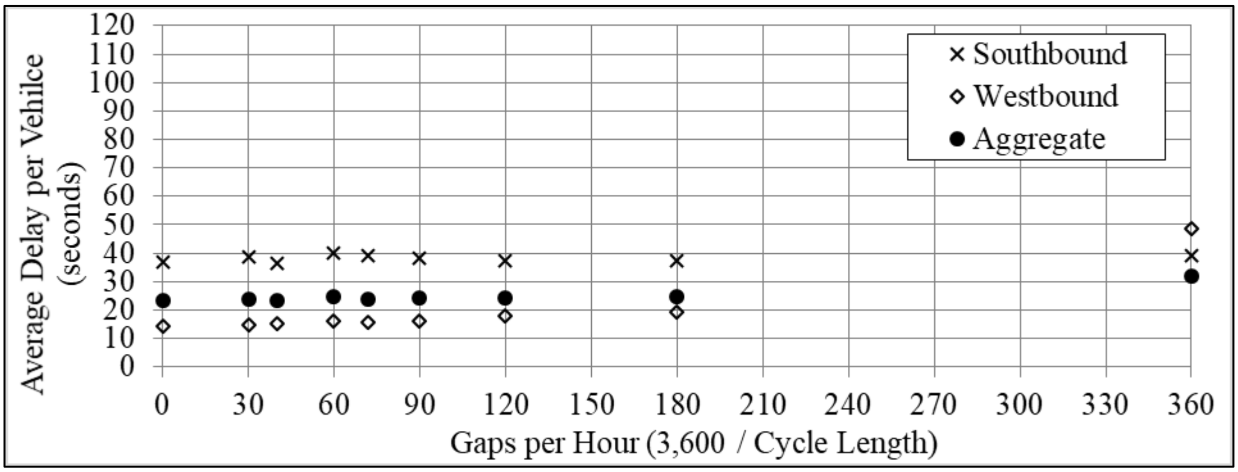
2b) shows the delay results where a 10-second red phase is set for the signal (two vehicles may exit per gap).

As the number of artificial gaps per hour increases, the pattern of average delay per vehicle is generally the same for both the 5-second and 10-second gap models. As gaps-per-hour increases, the average delay on the westbound approach increases while the average delay on the southbound approach decreases. Importantly, the decrease on the southbound approach delay is greater than the increase on the westbound approach delay, with provides an overall improvement to the aggregate delay. This is consistent with the theoretical benefits of the metering signal anticipated. Delay (and capacity) is successfully transferred from the under-capacity westbound approach to the overcapacity southbound approach, when examined using a simplified model with only volume on the southbound and westbound approaches. The challenge then, is to see the impact of the artificial gaps on a more realistic traffic condition where volume on all four approaches already provide natural gaps in the system.

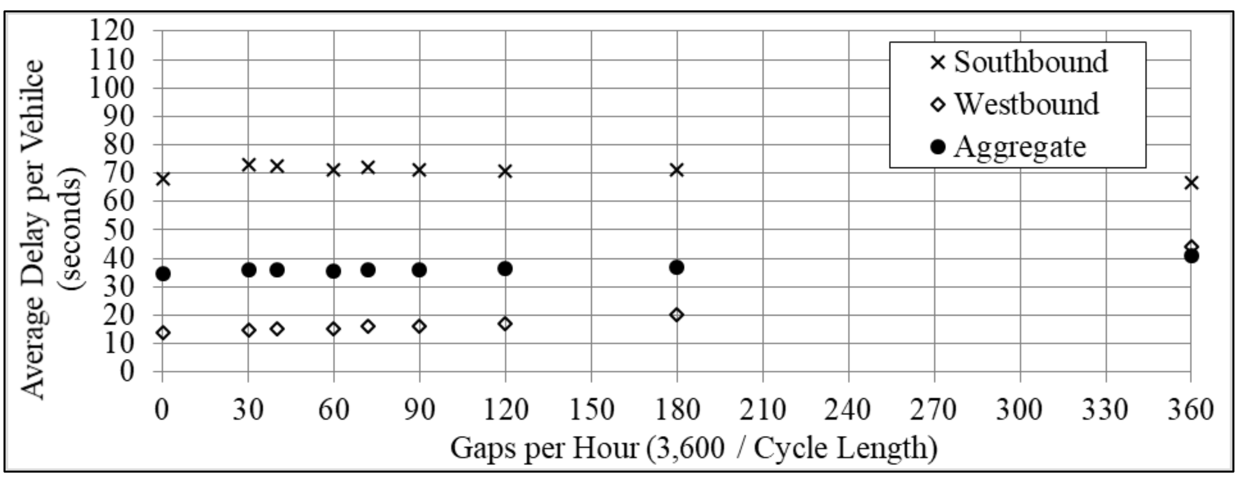
4.6.2 Second Investigation Results – Volumes on All Approaches

The second investigation includes complete volume scenarios having peak-hour directional flow as previously shown in **Table 4-1**. Three analyses are included, with **Figure 4-3** and **Figure 4-4** varying the volumes both for the southbound and other approaches, and **Figure 4-5** expanding the analysis to include a 10-second red phase.

The results shown below in **Figure 4-3** examine the sensitivity of releasing a southbound approach by installing a metering signal on the westbound approach of a single-lane roundabout, simulating traffic on all four approaches, and a 5-second red phase (one vehicle may exit per gap). **Figure (4-3a)** shows the delay results with 700 veh/hr flowrate southbound and 500 veh/hr flowrate on all other approaches. **Figure (4-3b)** shows the delay results with 800 veh/hr flowrate southbound and 500 veh/hr flowrate on all other approaches.



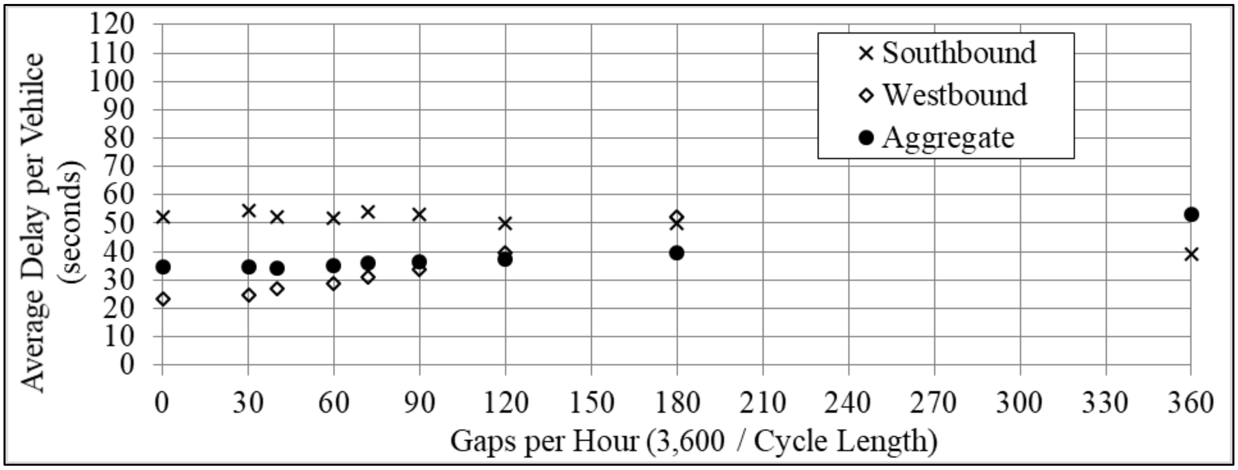
(a) 5-second red phase per cycle, 700 veh/hr southbound and 500 veh/hr all others



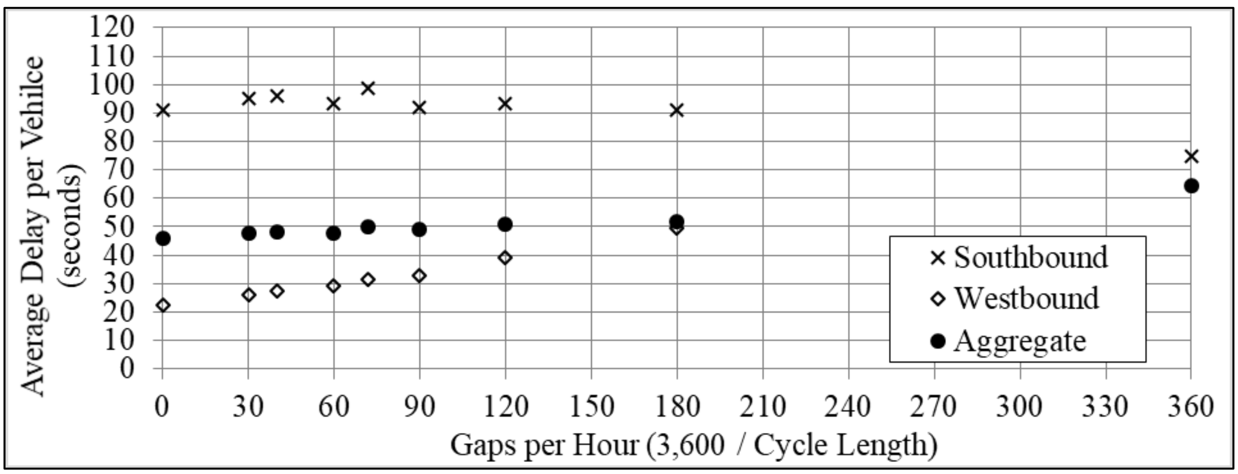
(b) 5-second red phase per cycle, 800 veh/hr southbound and 500 veh/hr all others

Figure 4-3 Delay implications for signal timing of metered roundabouts – full volume part 1

The results shown below in **Figure 4-4** continue to examine the sensitivity of releasing a southbound approach by installing a metering signal on the westbound approach of a single-lane roundabout, simulating traffic on all four approaches, and a 5-second red phase (one vehicle may exit per gap). **Figure (4-4a)** shows the delay results with 700 veh/hr flowrate southbound and 600 veh/hr flowrate on all other approaches. **Figure (4-4b)** shows the delay results with 800 veh/hr flowrate southbound and 600 veh/hr flowrate on all other approaches.



(a) 5-second red phase per cycle, 700 veh/hr southbound and 600 veh/hr westbound

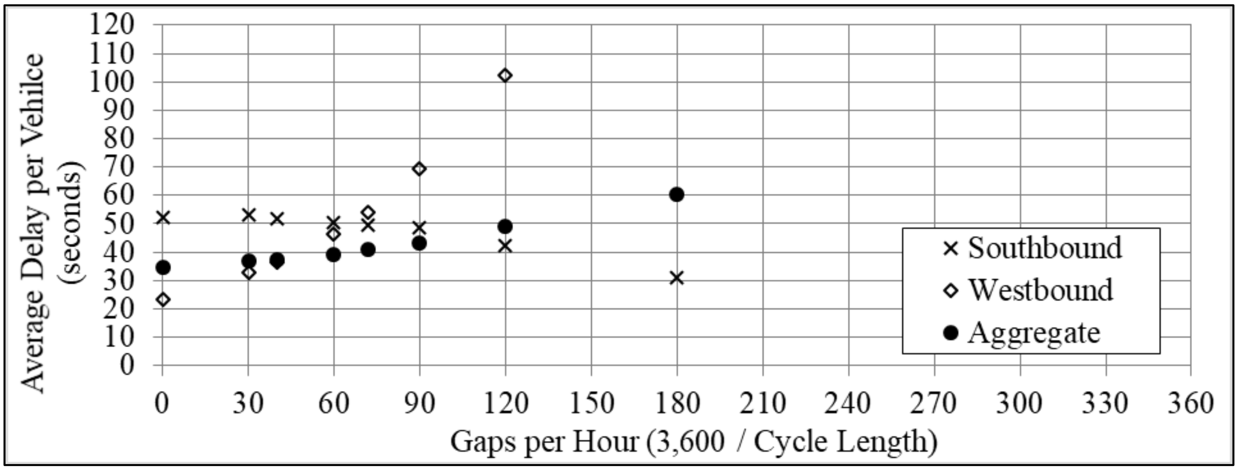


(b) 5-second red phase per cycle, 800 veh/hr southbound and 600 veh/hr westbound

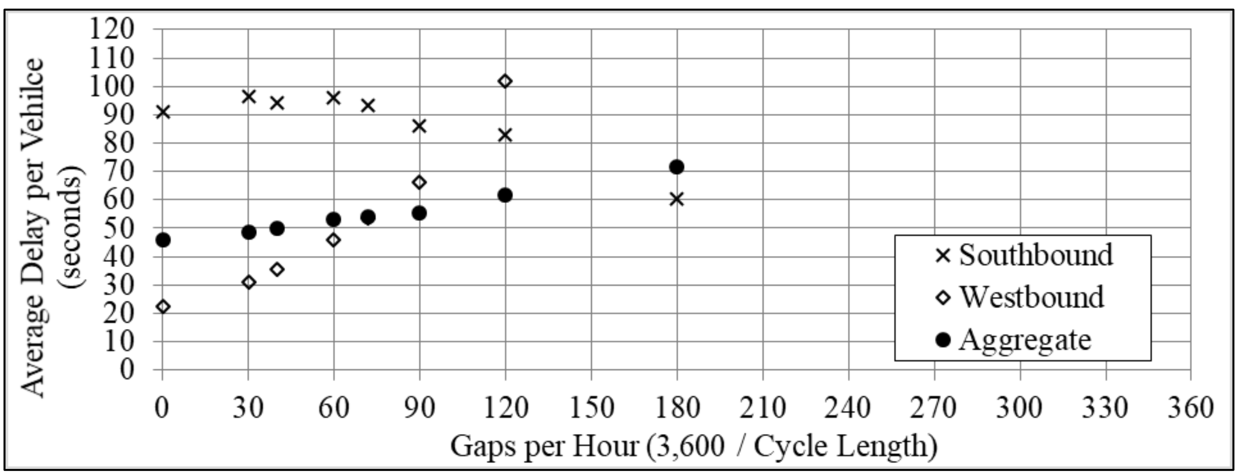
Figure 4-4 Delay implications for signal timing of metered roundabouts – full volume part 2

Finally, the results shown below in **Figure 4-5** continue to examine the sensitivity of releasing a southbound approach by installing a metering signal on the westbound approach of a single-lane roundabout, simulating traffic on all four approaches, with a 10-second red phase (one vehicle may exit per gap). **Figure (4-5a)** shows the delay results with 700 veh/hr flowrate southbound and 600 veh/hr flowrate on all other approaches.

Figure (4-5b) shows the delay results with 800 veh/hr flowrate southbound and 600 veh/hr flowrate on all other approaches.



(a) 10-second red phase per cycle, 700 veh/hr southbound and 600 veh/hr westbound



(b) 10-second red phase per cycle, 800 veh/hr southbound and 600 veh/hr westbound

Figure 4-5 Delay implications for signal timing of metered roundabouts – full volume part 3

Unfortunately, the results from the two-volume scenario in the first investigation were not duplicated in the full-volume scenario from the second investigation. In all six charts included in **Figure 4-3**, **Figure 4-4**, and **Figure 4-5**, increasing the number of artificial gaps per hour also increases the overall average delay for the roundabout facility. In particular, the positive change in delay to the westbound approach is seen to increase faster than the negative change in delay to the southbound approach.

The initial increases in delay on the southbound approach, between 30 and 90 gaps per hour, indicate that modeling issues may be at fault for the results more than a failure of the metering concept itself.

4.7. Research Limitations

While the results of the full-volume scenarios do not replicate the benefits of the metering approach to signalizing roundabouts seen in the initial investigation, there is some indication that the issue may be related to calibration of the driver behavior in the model, and not related to the failure of the application itself. In particular, all six charts from the second investigation indicate that, for long cycle lengths (producing fewer gaps per hour), there is an increase in the average delay of southbound vehicles relative to the unsignalized approach. This increase indicates that there are issues with the model and not with the metering itself, as it is highly unlikely that placing a red signal on the westbound approach could increase delay on the southbound approach. While it is possible that the artificial gaps created by the metering signal would somewhat align with the conflicting northbound movement creating regular gaps, it is not possible that the creation of artificial gaps on the westbound approach could result in fewer gaps available for the southbound movement. Further investigation should pursue refined calibration of the gap acceptance and follow-up headway behavior in the VISSIM traffic microsimulation model. If the utility of signal metering at roundabouts is to be assessed robustly, the models must provide logical results.

4.8. Conclusions and Recommendations

The primary focus of this research investigates the impact of metering signals on single-lane roundabouts. During peak-hours, some roundabouts experience a directional flow of traffic movement which leads to extensive delay and performance failure, as there are insufficient gaps to allow vehicles at that entry point to enter the circulating flow of the roundabout. By placing a metering signal on the adjacent upstream approach, artificial gaps can be introduced to the circulating flow, transferring latent capacity from one approach to a downstream approach that is operating at or above capacity.

To assess the hypothesis, two investigations were pursued. The first investigation assesses the results of metering by only considering volume scenarios with the controlling and controlled approaches. The results of this first investigation are consistent with the anticipated results from the metering of a roundabout approach, that the creation of artificial gaps from one traffic stream successfully transferred capacity to the downstream approach. As the number of gaps per hour that are artificially created increases, the delay of the controlling approach decreases, while the delay of the controlled approach increases more slowly, with an overall benefit to average delay per vehicle for the facility.

The second investigation assesses the effect of the metering signal on a single lane roundabout having a directional flow with full-volume scenarios. With the southbound movement used as the controlling approach experiencing the higher demand flowrates, the westbound approach becomes the controlled approach that receives the metering signal.

Unfortunately, the results of the second investigation showed unrealistic output which strongly contradicted the theoretical approach results that were found in the first investigation. The analysis indicated that on placing the metering signals on the westbound approach, the delay on the southbound approach becomes unstable, and in some cases was observed to increase. While the results of the second investigation do not support the theoretical benefits of the metering signal, the author is concerned that these results are strongly biased with modeling issues that remain to be resolved.

CHAPTER 5 CONCLUSION

This research examines two areas of study related to the operational analysis of roundabouts. The first examines the reliability of using the critical sum method (CSM) algorithmic formulation, a capacity based approach, for predicting the anticipated HCM average control delay at a single lane roundabout. The second investigates the impact of metering signals on a single lane roundabout in order to consider their potential as a solution for problems caused by directional traffic flow. The findings of these two investigations are summarized here, along with recommendations for future research.

5.1. Assessment of the Critical Sum Method

In order to be a successful planning tool for transportation practitioners, the critical sum method must determine whether the performance of a roundabout is close to failure, failing, or beyond failure. For assessing the effectiveness of this method, 250,000 volume scenario combinations with the randomized parameters were analyzed.

To study the relationship a graphical investigation was conducted, assessing how the average value of delay increases as the critical sum (worst-case approach) increases. A growing scatter in the delay values was observed once the critical sum values exceeded 900 veh/ln/hr. The option of using the weighted critical sum instead of the maximum was also explored, however, the same scatter was observed. Using engineering judgement, it was determined that for the critical sum method to be reliable, 95% of the delay values should fall with in ±5 seconds of the predicted mean value of the delay. The results indicated that once the critical sum value of 1,000 veh/ln/hr or higher was reached, the delay values become widely spread and it not meet the reliability criteria set for use as an indicator variable. Further exploration was conducted which indicated that additional factors such as the directional split of traffic or the turn movement percentages had negligible impact on the reliability of the critical sum method.

Ultimately, the research indicated that the critical sum method would confidently anticipate whether the level of service (LOS) for a single lane roundabout is an A or a B as per the Highway Capacity Manual (HCM) [7], average delay standards for unsignalized intersections, however, it does not indicate if the roundabout will be operating at LOS D, E, or F to identify whether its failing in performance or close to failing . As a result, not enough evidence was found from the data, to indicate that the critical sum method is a reliable tool to predict the average control delay for a single lane roundabout.

5.2. Analysis of Roundabout Metering Signals

During peak hours, some roundabouts experience a directional flow of traffic movement which leads to extensive delays, as there are insufficient gaps to allow vehicles at that entry point to enter the circulating flow of the roundabout. By placing a metering signal on the adjacent upstream approach, artificial gaps can be introduced to the circulating flow, transferring latent capacity from one approach to a downstream approach that is operating at or above capacity.

To assess this hypothesis, two investigations were pursued with the microsimulation software VISSIM. The models were calibrated to provide results consistent with the HCM algorithmic equations [7]. The first investigation assessed the metering by only considering volume scenarios with the controlling and controlled approaches. The results of this first investigation are consistent with the anticipated results of the metering of a roundabout approach, that the creation of artificial gaps from one traffic stream successfully transferred capacity to the downstream approach.

The second investigation assessed the effect of a metering signal on a single lane roundabout having a directional flow with full-volume scenarios. With the southbound movement used as the controlling approach experiencing the higher demand flow rates, the westbound approach became the controlled approach receiving the metering signal. Unfortunately, the results of the second investigation did not substantiate the results from the investigation with traffic on only two approaches. Sensitivity analysis was done twice, with a 5 and 10 second red phase in order to indicate the impact of the red phase interval. No significant difference was observed from the results, and both outputs followed similar patterns of behavior. This increase indicates that there may be issues with the model and not with the metering itself, as there is highly unlikely that placing a red signal on the westbound approach could increase delay on the southbound approach. Several volume scenarios were analyzed. Six scenarios are presented in **Figure 4-3**, **Figure 4-4**, and **Figure 4-5** in Chapter 4 and the rest are presented in Appendix A and Appendix B.

5.3. Recommendations for Future Research

Future research on the operational analysis of the roundabouts could be conducted by further exploring the reliability and efficiency of using the critical sum method as an accurate delay predictor for other facilities. Research-based validations would be required to obtain robust findings. Subsequently, other parameters than the ones studied in this research could be explored for identifying the reliability of the critical sum approach, such as roundabout geometry, gap acceptance, driver behavior, etc.

As the calibrated factors (minimum gap and headway) for roundabout metering signal analysis may be confounding the results for the full-volume metering signal analysis, further studies could be conducted to indicate the appropriate calibration values which could reflect more accurate behavior. Additionally, another roundabout operational analysis software package such as SIDRA could be utilized for comparing its results with the VISSIM output.

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driver's critical headway," *Transp. Res. Board, Part A*, 1999.

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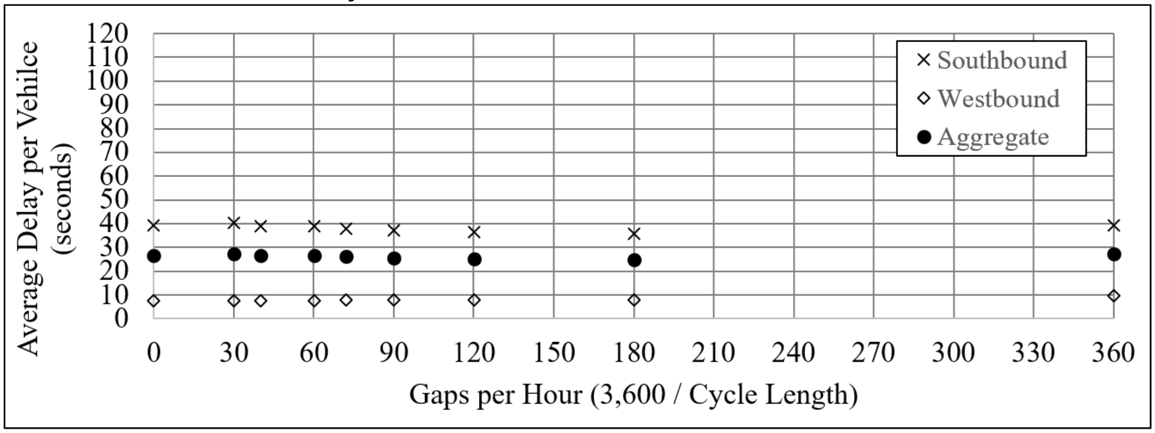
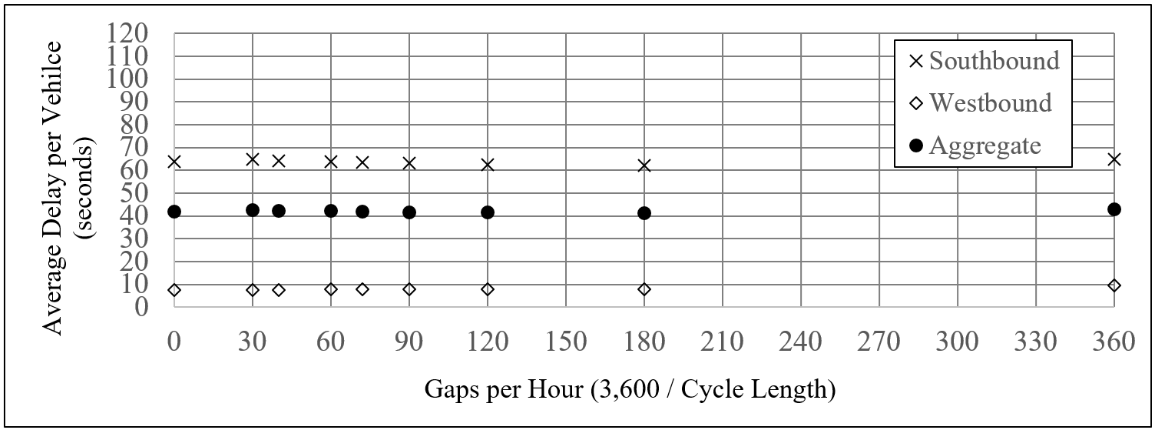
APPENDIX A

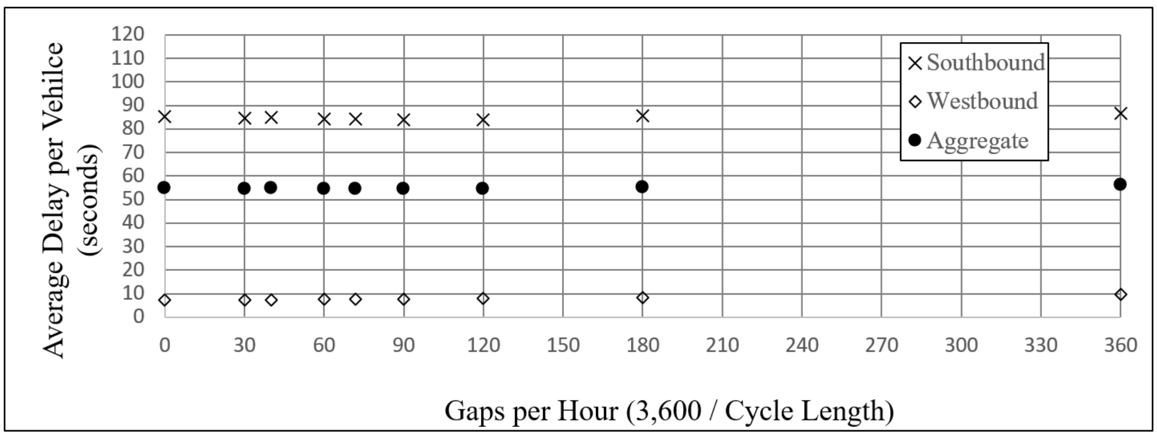
5-Seconds Red Phase Analysis



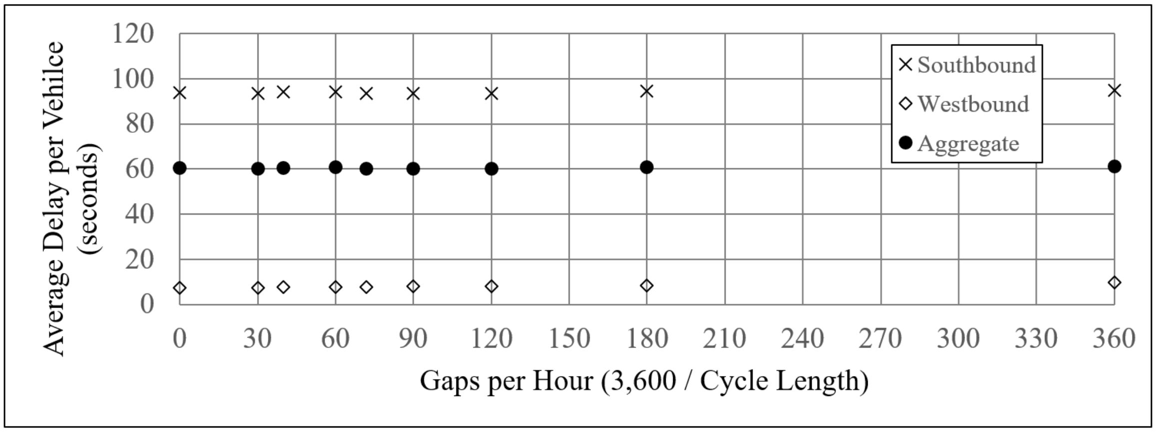
(a) 5-second red phase per cycle, 900 veh/hr southbound and 200 veh/hr all others



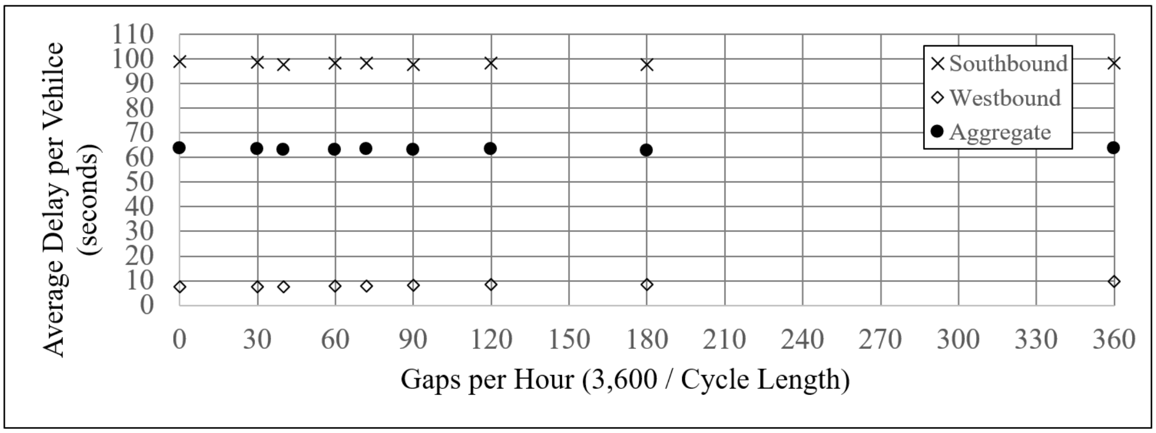
(b) 5-second red phase per cycle, 1000 veh/hr southbound and 200 veh/hr all others



(c) 5-second red phase per cycle, 1100 veh/hr southbound and 200 veh/hr all others

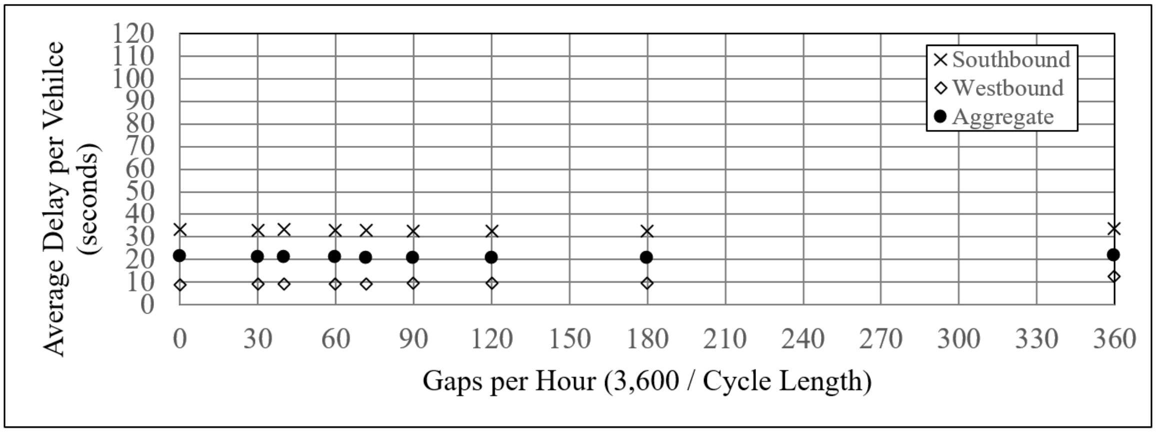


(d) 5-second red phase per cycle, 1200 veh/hr southbound and 200 veh/hr all others

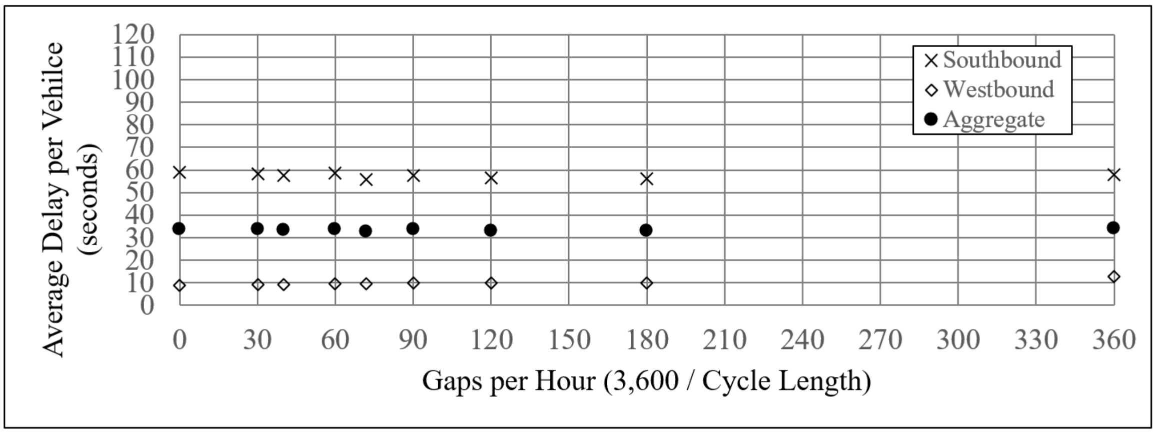


(e) 5-second red phase per cycle, 1300 veh/hr southbound and 200 veh/hr all others

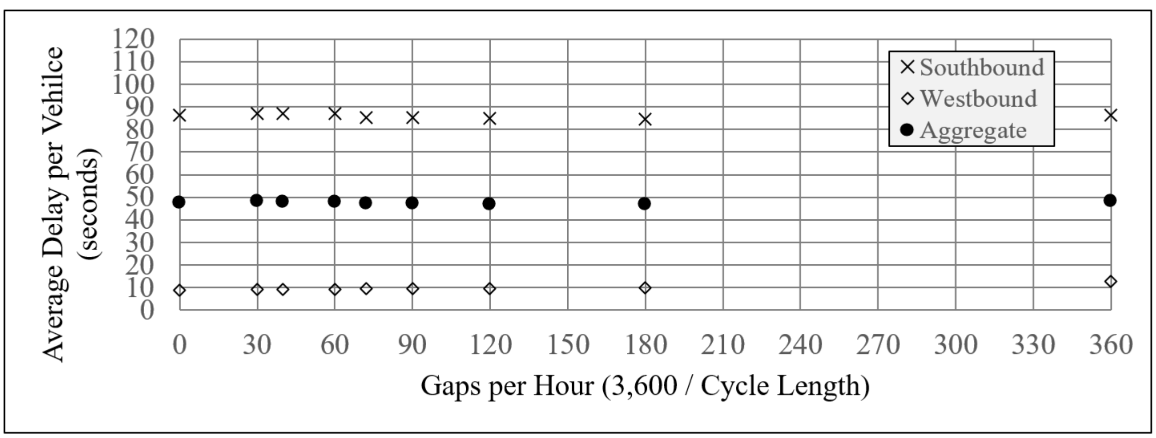
Figure 1 Delay implications for signal timing of metered roundabouts – full volume



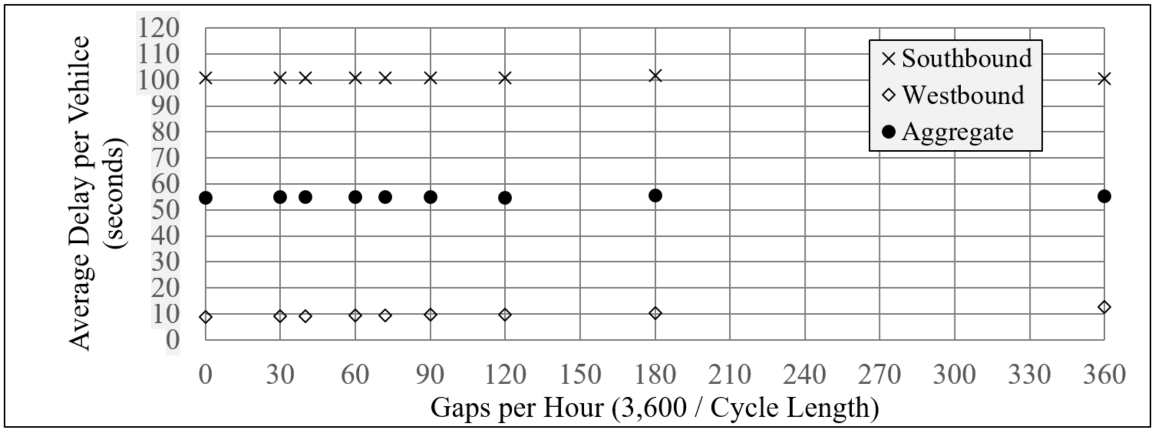
(a) 5-second red phase per cycle, 800 veh/hr southbound and 300 veh/hr all others



(b) 5-second red phase per cycle, 900 veh/hr southbound and 300 veh/hr all others

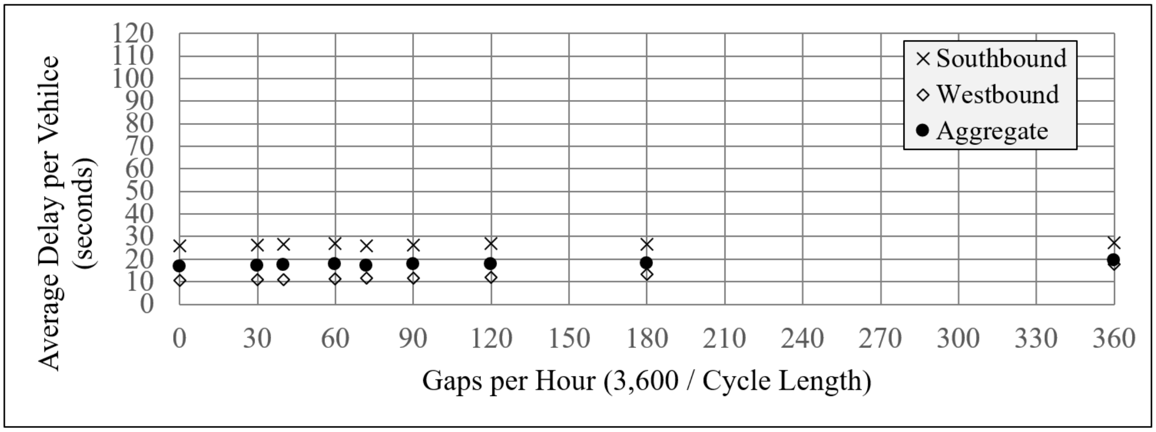


(c) 5-second red phase per cycle, 1000 veh/hr southbound and 300 veh/hr all others

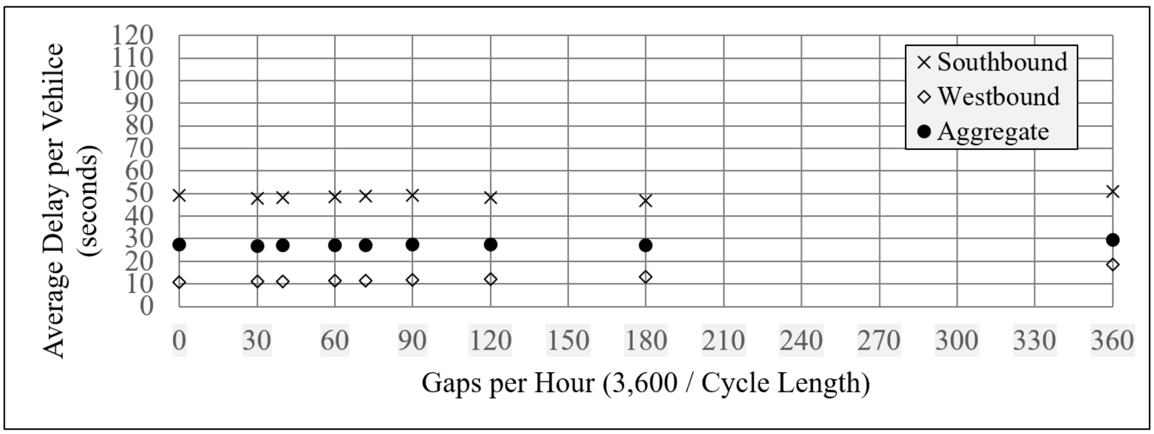


(d) 5-second red phase per cycle, 1100 veh/hr southbound and 300 veh/hr all others

Figure 2 Delay implications for signal timing of metered roundabouts – full volume



(a) 5-second red phase per cycle, 700 veh/hr southbound and 400 veh/hr all others



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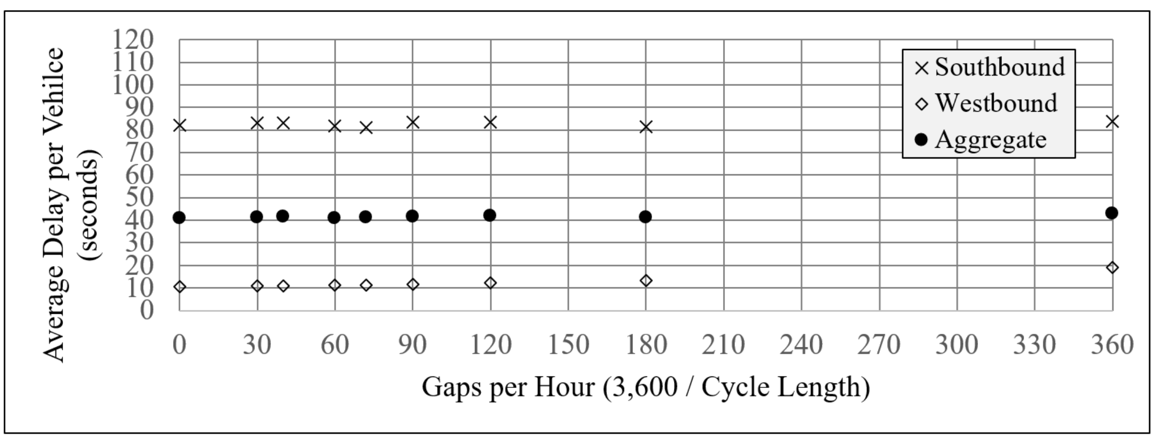
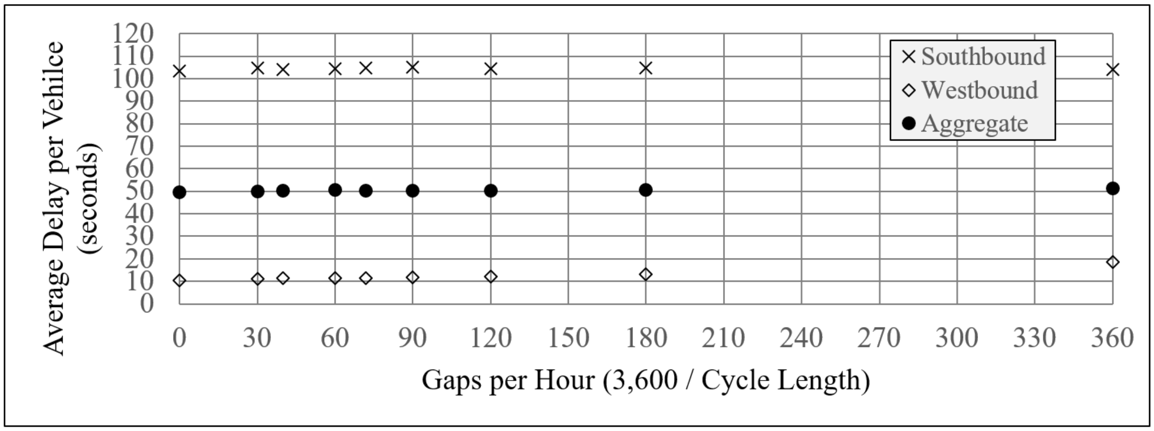
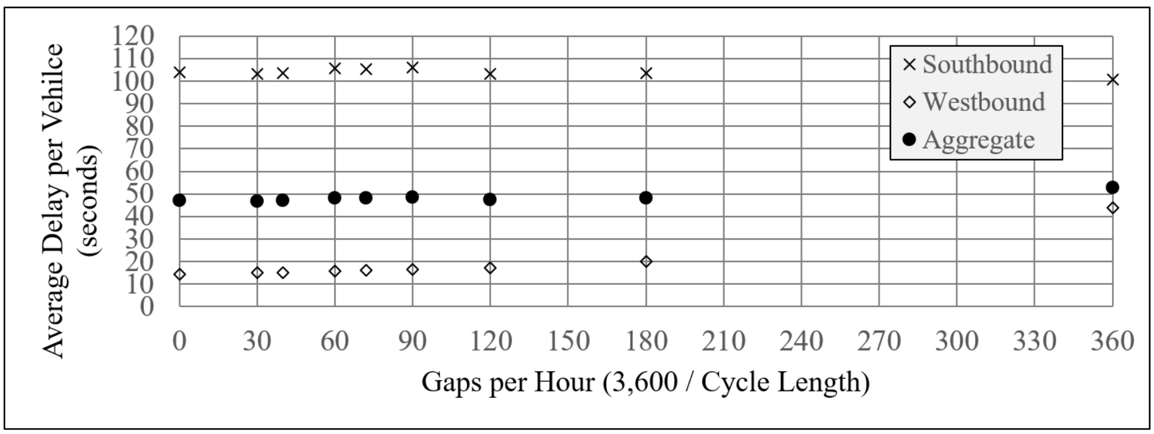


(c) 5-second red phase per cycle, 900 veh/hr southbound and 400 veh/hr all others



(d) 5-second red phase per cycle, 1000 veh/hr southbound and 400 veh/hr all others

Figure 3 Delay implications for signal timing of metered roundabouts – full volume

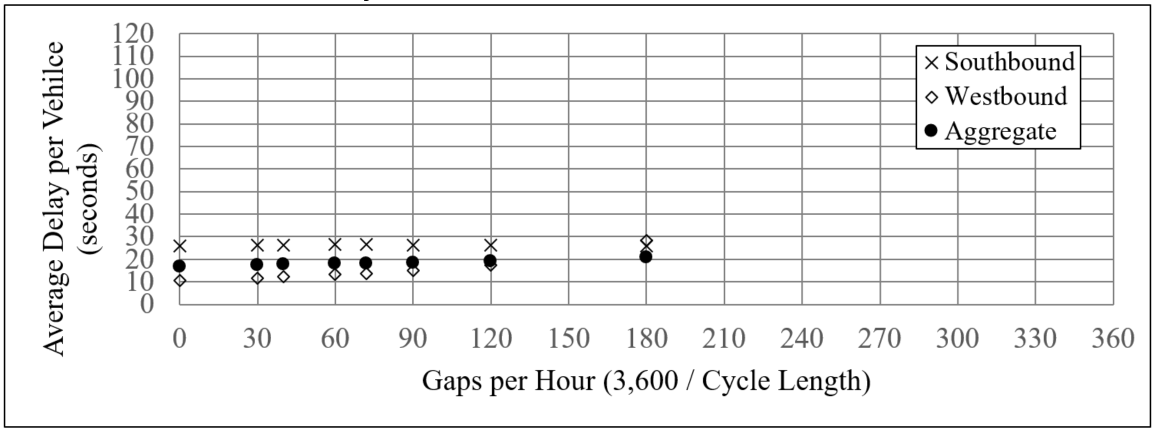


(a) 5-second red phase per cycle, 900 veh/hr southbound and 500 veh/hr all others

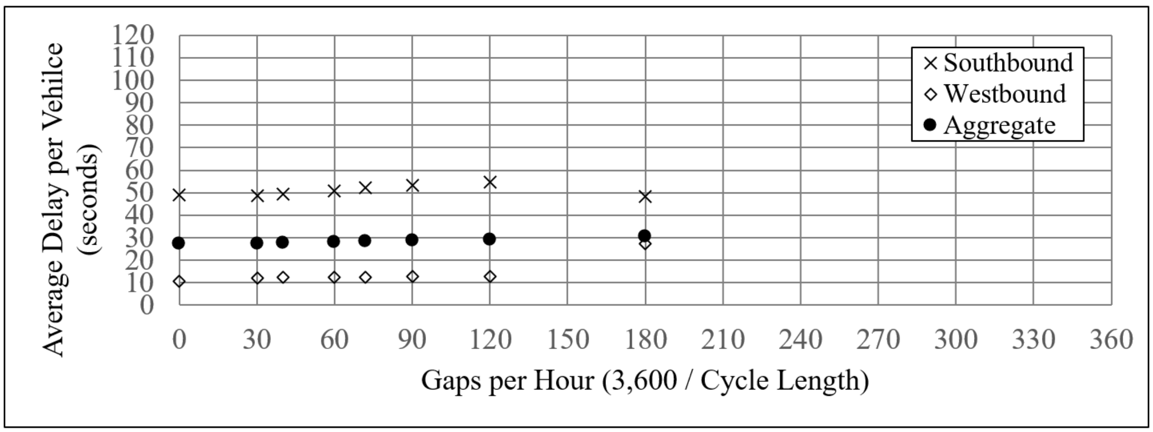
Figure 4 Delay implications for signal timing of metered roundabouts – full volume

APPENDIX B

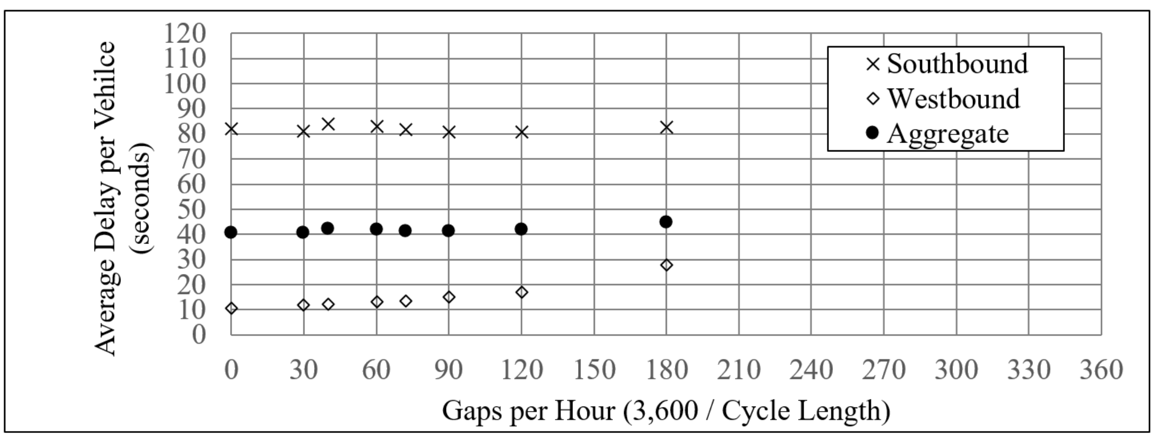
10-Seconds Red Phase Analysis



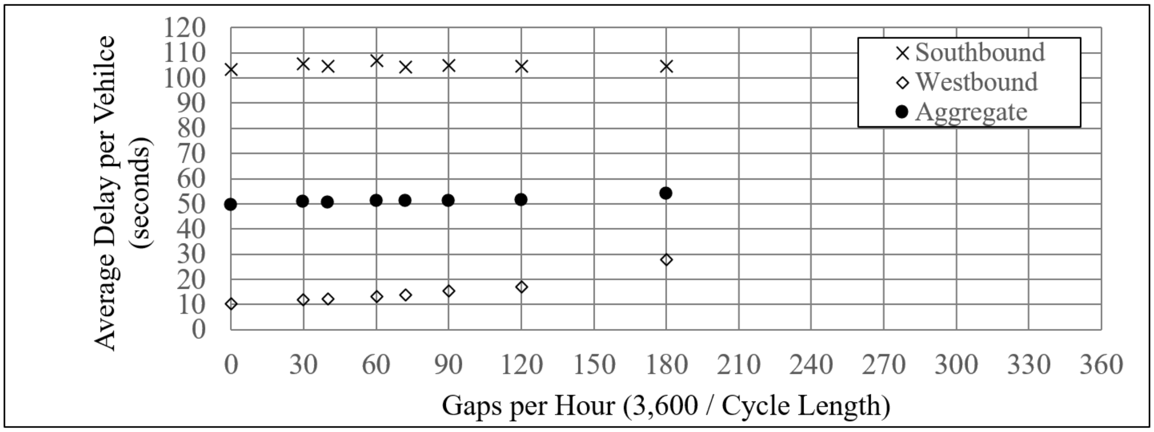
(a) 10-second red phase per cycle, 700 veh/hr southbound and 400 veh/hr all other



(b) 10-second red phase per cycle, 800 veh/hr southbound and 400 veh/hr all other

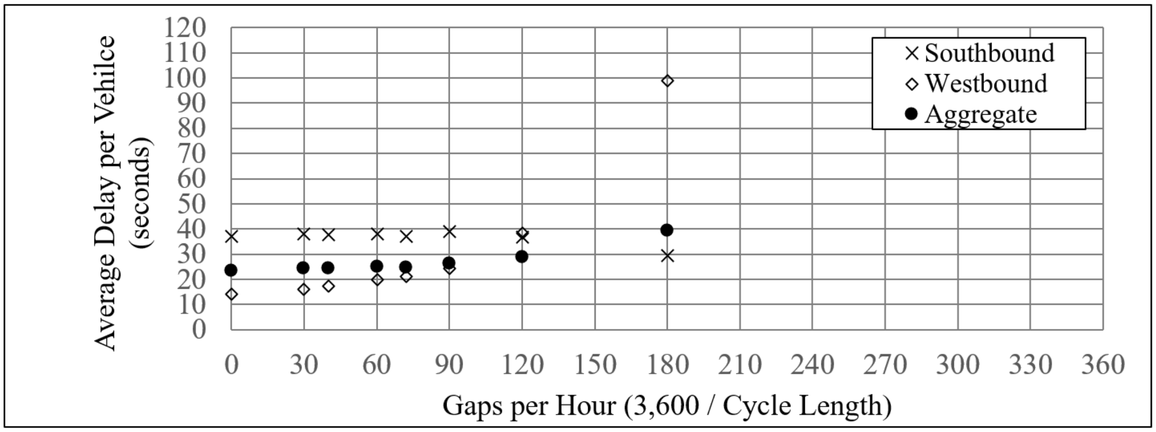


(b) 10-second red phase per cycle, 900 veh/hr southbound and 400 veh/hr all other

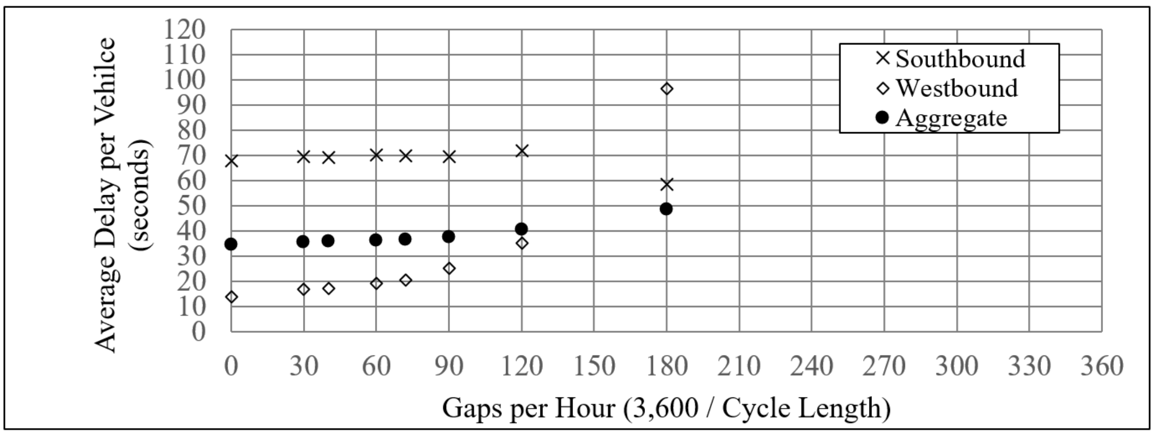


(d) 10-second red phase per cycle, 1000 veh/hr southbound and 400 veh/hr all other

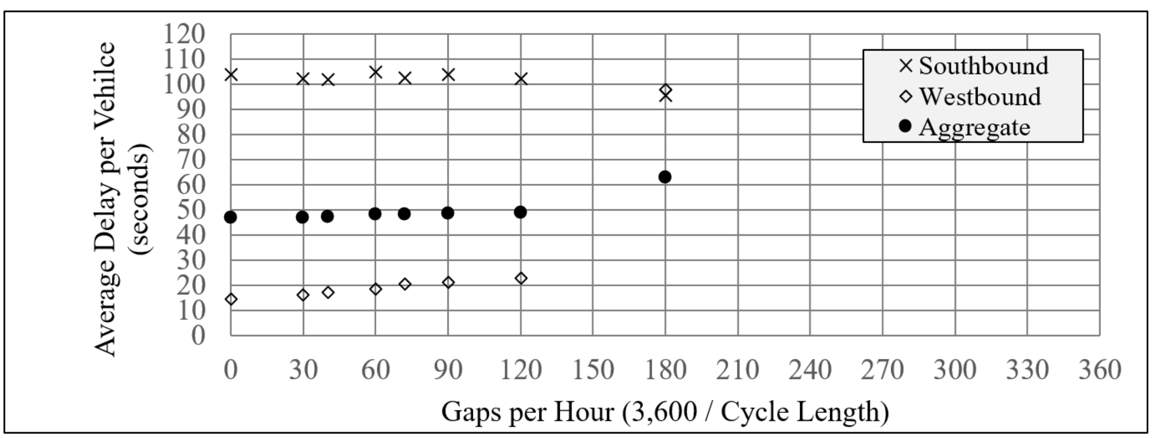
Figure 5 Delay implications for signal timing of metered roundabouts – full volume



(a) 10-second red phase per cycle, 700 veh/hr southbound and 500 veh/hr all other



(b) 10-second red phase per cycle, 800 veh/hr southbound and 500 veh/hr all other



(c) 10-second red phase per cycle, 900 veh/hr southbound and 500 veh/hr all other

Figure 6 Delay implications for signal timing of metered roundabouts – full volume