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CALIBRATION OF THE HCM 2010 SINGLE-LANE ROUNDABOUT CAPACITY EQUATIONS FOR GEORGIA CONDITIONS

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CALIBRATION OF THE HCM 2010 SINGLE-LANE ROUNDABOUT CAPACITY EQUATIONS FOR GEORGIA CONDITIONS

By

Michael Hunter, Ph.D. (P.I.)

Michael O. Rodgers, Ph.D. and Justin Bansen (Co-PI)

Laura Schmitt, Christina Barry, and Wonho Suh, Ph.D.
(Graduate Research Assistants)

School of Civil and Environmental Engineering and
Georgia Institute of Technology

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16. Abstract:

The objective of this study was to provide the Georgia Department of Transportation with calibrated operational performance models for single-lane roundabouts in Georgia. Some of the pertinent conclusions from this study are:

1. The HCM 2010 default values for critical and follow-up headway are 3.200 and 5.000 seconds, respectively. This study found the critical and follow-up headway for an analysis excluding exiting vehicles is 4.747 seconds and 3.265 seconds, respectively. This study's calibrated model excluding exiting vehicles predicts higher capacity than the 2010 HCM model except at conflicting volumes of 300 vehicles per hour or less.
2. Exiting vehicles impact the entry capacity. Specifically, the percentage of conflicting vehicles that are exiting vehicles influence the change in the headway values and thus change the predicted entry capacity.
3. This study also calculated the follow-up and critical headway values for sites with consistent modern roundabout features. Six of the thirteen roundabouts this study collected data at were considered modern roundabouts. There is less than a 0.1 seconds difference between the headway values of just the modern roundabouts and all the roundabouts. These results suggest that the geometric differences between the modified list and remaining roundabouts do not affect the operations of the roundabout.

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LIST OF SYMBOLS

$c_{e,pce}$	Capacity of the approach lane under consideration in pce
n	Number of observations
t	Gap or lag
Δt	Adjustment time
T_1	Leading time stamp
T_2	Time stamp of the following circulating vehicle
t_c	Critical headway
t_f	Follow-up headway
$v_{c,pce}$	Conflicting flow in Passenger Car Equivalents (PCE)
v/c	Volume-to-capacity ratio

LIST OF ABBREVIATIONS

AADT	Average annual daily traffic
ALP01-SB	Alpharetta southbound
COL01-SEB	Columbus southeastbound
COL01-SWB	Columbus southwestbound
COV01-SB	Covington southbound
COV01-NB	Covington northbound
COV01-SB	Covington southbound
COV01-WB	Covington westbound
COV01-EB	Covington eastbound
CSV	Comma separated values
DOU01-EB	Douglasville eastbound
DOU01-WB	Douglasville westbound
DOU01-SB	Douglasville southbound
DOUL01-EB	Duluth eastbound
EMO01-SEB	Emory southeastbound
FAY01-EB	Fayetteville eastbound
FAY01-SB	Fayetteville southbound
FAY01-NB	Fayetteville northbound
FHWA	Federal Highway Administration
GDOT	Georgia Department of Transportation
HCM	Highway Capacity Manual
HIN01-WB	Hinesville westbound
HIN01-SB	Hinesville southbound
HOL01-EB	Holly Springs eastbound
HOL01-NB	Holly Springs northbound
LOS	Level of service
NCHRP	National Cooperative Highway Research Program
NEW01-EB	Newnan eastbound
NEW01-WB	Newnan westbound
pce	passenger car equivalents
pcu	passenger car units
PHF	Peak hour factor
QA/QC	Quality Assurance/Quality Control
ROS01-EB	Roswell eastbound (5/15/12)
ROS01-SWB	Roswell southwestbound (5/15/12)
ROS02-EB	Roswell eastbound (10/23/12)
ROS02-SWB	Roswell southwestbound (10/23/12)
STARS	State Traffic and Report Statistics
std. dev.	Standard Deviation

STS01-WB	St. Simons westbound
STS01-EB	St. Simons eastbound
TOPS	Wisconsin Traffic Operations and Safety
URA	Undergaduate research assistant
VIL01-SWB	Villa Rica southwestbound

SUMMARY

Roundabouts are becoming an increasingly appealing alternative intersection treatment because of their safety and efficiency benefits. These benefits are sufficiently large that the Georgia Department of Transportation (GDOT) requires a roundabout be considered for every new or reconstructed intersection. One aspect of determining if a roundabout is a feasible intersection treatment is to perform an operational analysis through the use of one or more intersection capacity models. To assist in these analyses, GDOT developed a Roundabout Analysis Tool that incorporates two different single-lane roundabout capacity models. The first of these models is identical to the default single-lane roundabout capacity equation found in the 2010 Highway Capacity Manual (HCM) while the second model uses the same capacity equations with different parameter values calibrated with follow-up and critical headway values derived from studies in California and Bend, Oregon. GDOT current suggests use of the first, more conservative, model based on the belief that drivers in Georgia are generally not as familiar with roundabouts as drivers in Oregon or California. To examine the validity of this assumption and to provide improved capacity predictions for existing and proposed Georgia roundabouts, a local (Georgia) calibration of the 2010 HCM roundabout capacity equation, including collection of the field data necessary to calculate follow-up and critical headways at Georgia roundabouts, was undertaken through this study. This study also evaluated the impact of including vehicles exiting the roundabout immediately before reaching the conflict point in the volume calculations. These vehicles are not considered in the current models.

This study used the methodology outlined in the NCHRP Report 572 as a guide. In order to measure follow-up and critical headway, operations at Georgia roundabouts were recorded with video cameras. The research team filmed 28 approaches at thirteen

Georgia roundabouts for a total of 56.5 hours. The video was processed semi-automatically using an in-house computer program. Based on keystrokes entered by the analysis during a review of the recorded video, the event timestamps necessary for calculating follow-up and critical headways were recorded. The NCHRP Report 572 presents several methods for calculating both follow-up and critical headway using these recorded values. Follow-up headway was calculated using both the "queued data" and the "move-up time" methods while Critical headway was calculated using three methods (Method 1, 2, and 3) presented in NCHRP Report 572. The results from these analyses are presented in this report. Based on an evaluation of these results, the final recommended values of Critical and follow-up headways were those found using the "move-up time" and "method 2" approaches respectively.

The follow-up and critical headway values were calculated for two different data sets: 1) including exiting vehicles and 2) excluding exiting vehicles. The critical and follow-up headway for an analysis including exiting vehicles is 4.192 seconds and 2.788 seconds, respectively. The critical and follow-up headway for an analysis excluding exiting vehicles is 4.747 seconds and 3.265 seconds, respectively. This study found that the calibrated model excluding exiting vehicles predicts higher capacity than the 2010 HCM model that GDOT recommends which also excludes exiting vehicles. In addition, this study found that including the exiting vehicles impacts the capacity. The capacity increases or decreases based on the percentage of conflicting vehicles that are exiting vehicles.

CHAPTER 1: INTRODUCTION

The modern roundabout was developed in Great Britain in the 1960's in response to safety and efficiency issues of traffic circles but the first roundabout constructed in the United States was not installed until 1990 in Summerlin, Nevada [1, 4]. The 2010 Highway Capacity Manual (HCM 2010) defines roundabouts as "intersections with a generally circular shape characterized by yield on entry and circulation around a central island (counterclockwise in the United States)" [2]. Figure 1 shows the general characteristics of a roundabout.

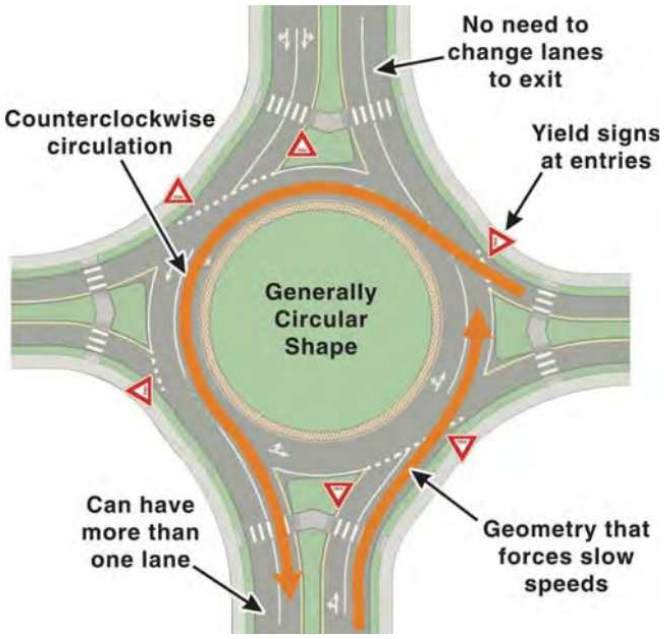

Figure 1. Characteristics of a modern roundabout [3]

As a relatively new intersection treatment, several in-depth reports have been published to enhance knowledge in roundabout operations and design. In 2000, the Federal Highway Administration (FHWA) published *Roundabouts: An Informational Guide* (FHWA-RD-00-067) which includes guidelines for the planning phase, operational analysis, and design of roundabouts [5]. However, the report was primarily based on

European and Australian data as at the time there were only 38 roundabouts in the United States [3]. Despite this limitation the documented benefits and outlined design guidelines of the 2000 FHWA report helped prompt the construction of many additional roundabouts in the United States.

With the increasing US interest in roundabouts in 2007 *NCHRP Report 572: Roundabouts in the United States* was published. This document presented the safety and operational benefits of the newly expanded United States roundabout inventory. The report analyzed 55 sites before and after the installation of a roundabout. Roundabouts were seen to be successful in a wide variety of environments in the United States including urban, suburban, and rural [1]. For instance, it was found that the estimated reduction in all crash and injury crashes were 35.4% and 75.8%, respectively [1]. Building on these efforts in 2010 the first roundabout informational guide was updated in *NCHRP Report 672 Roundabouts: An Informational Guide – Second Edition.*

1.1 SAFETY COUNTERMEASURE

In 2008, the FHWA released a Guidance Memorandum on Consideration and Implementation of Proven Safety Countermeasures, identifying roundabouts as one of nine safety countermeasures recognized and supported by FHWA [6]. This document states [6]:

> *Roundabouts are the preferred safety alternative for a wide range of intersections. Although they may not be appropriate in all circumstances, they should be considered as an alternative for all proposed new intersections on federally-funded highway projects, particularly those with major road volumes less than 90 percent of the total entering volume. Roundabouts should also be considered for all existing intersections that have been identified as needing major safety or operational improvements. This would include freeway interchange ramp terminals and rural intersections.*

The Georgia Department of Transportation (GDOT) also identifies roundabouts as the preferred safety alternative for intersections. As outlined in Chapter 8 of the GDOT Design Policy Manual [7],

> *a roundabout shall be considered in the following situations: for any intersection being designed on a new location or to be reconstructed; for any existing intersection that has been identified as needing major safety or operational improvement (or where improvements are otherwise planned); and for all intersections where a request for a traffic signal has been made.*

Supported by these policies and growing knowledge in operations and design roundabouts have seen rapidly increasing interest and implementation in Georgia.

1.2 PROJECT OBJECTIVES

Determining the feasibility of a roundabout is, in part, based on an operational analysis. The operational performance of an existing or proposed roundabout is based on many factors including a capacity evaluation that can be assessed through application of appropriate capacity models. For example, the roundabout capacity models presented in NCHRP Report 572 are used in the 2010 HCM.

NCHRP Report 572 and the 2010 HCM equations were used to develop the GDOT Roundabout Analysis Tool which analyzes the performance of single and multi-lane roundabouts. Currently, the tool supports both a “national default” parameter (calibration) set and an additional set of calibration parameters based on data from California and Bend, Oregon [8]. The validity of the use of the estimated capacities to Georgia drivers and conditions was unknown and this project was undertaken to calibrate the 2010 HCM roundabout capacity equations based on Georgia conditions. Locally calibrated capacity predictions will assist in the decision making process to determine if a roundabout is a feasible option and, if so, select the appropriate roundabout configuration (single-lane roundabout or multi-lane roundabout).

This document is intended to:

1. Provide results of the calibrated HCM 2010 single-lane roundabout capacity equations for Georgia conditions and drivers based on locally measured follow-up and critical headway,
2. Provide a comparison of the impacts of the different NCHRP methods for calculating critical and follow-up headway,
3. Provide a comprehensive guide for data collection, data reduction, and data analysis that can be used for future replication of the calibration process,
4. Evaluate the impact of including exiting vehicles in the roundabout capacity analysis.

CHAPTER 2: LITERATURE REVIEW

In this chapter we will review the current state of practice for roundabouts in Georgia, as well as calibration data requirements, data collection methods and findings from previous calibration studies.

2.1 ROUNDABOUTS IN GEORGIA

In Georgia, roundabouts have been built by GDOT as well as other entities including counties, cities, and private developers [9]. As previously noted, FHWA has identified roundabouts as a preferred safety alternative. Similarly, GDOT "…considers roundabouts as the preferred safety alternative for a wide range of intersections on public roads" [7]. The first modern roundabout to be built in Georgia was constructed in Carroll County in the city of Whitesburg in 2000. Since then, roundabouts have been built in in a variety of rural, urban and suburban settings across Georgia and many additional roundabouts are currently in various stages of planning, design, or construction. While the majority of roundabouts in Georgia are single-lane roundabouts with either three or four legs there are several multi-lane roundabouts in Cherokee County as well as one in Glynn County on St. Simon's Island [9].

The GDOT Design Policy Manual outlines the steps for conducting a roundabout feasibility study and guidance on roundabout use. GDOT provides a Roundabout Analysis Tool in the form of a spreadsheet for analyzing the performance of roundabouts as a part of these feasibility studies [9]. This tool provides analysis for both single-lane and multi-lane roundabouts, for both the uncalibrated (i.e. default) version and a calibrated version of the HCM 2010 model based on data from California and the

City of Bend, Oregon. The uncalibrated version utilizes the default model parameters found in NCHRP Report 572. Neither of these models may correctly reflect Georgia conditions or drivers [8].

2.2 NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM (NCHRP) STUDIES

NCHRP Project 3-65 collected traffic data from several selected roundabouts in the United States between 2002 and 2004. The results of this study were initially presented at the first International Roundabout Conference in 2005 and are the data source for the 2010 HCM model used in the GDOT Roundabout Analysis Tool. In addition, these results formed the basis for much of the guidance included in the subsequent NCHRP 572 and NCHRP 672 reports [10]. In this study, an inventory of all known roundabouts in the United States at that time was compiled. Traffic data were collected from 31 of these roundabouts located in ten different states. The states included in the data collection for NCHRP Project 3-65 were: Colorado, Kansas, Maine, Maryland, Michigan, Nevada, Oregon, Utah, Vermont, and Washington. Unfortunately, none of these states was located in the Southeast. In the selection of roundabouts for data collection, the first criterion listed by the authors was, “The likelihood of finding continuous (persistent) queuing on one or more of the roundabout approaches, representing capacity conditions” [10]. Additionally, the project team considered the locations of the roundabouts and strove to choose both single-lane and multi-lane roundabouts with various geometric characteristics [10].

Mast-mounted video cameras were used for data collection at all sites. According to NCHRP Report 572, the equipment used for recording the roundabout operations included masts, digital cameras, omni-directional cameras, and DVD recorders [10]. The data collection team consisted of four people.

The roundabout traffic data collected were used in existing capacity equations to determine how well these equations estimated the capacity of American roundabouts. Models from the United Kingdom, Australia, Germany, France, and Switzerland, were compared to those from the then current HCM 2000 and FHWA models. For almost all single-lane roundabouts at which data were collected the models predicted higher capacities than were actually observed. Similar results were noted for multi-lane roundabouts with the exception of the HCM 2000 model [1]. Figure 2 illustrates the results from the various models and the observational data for one of the roundabouts.

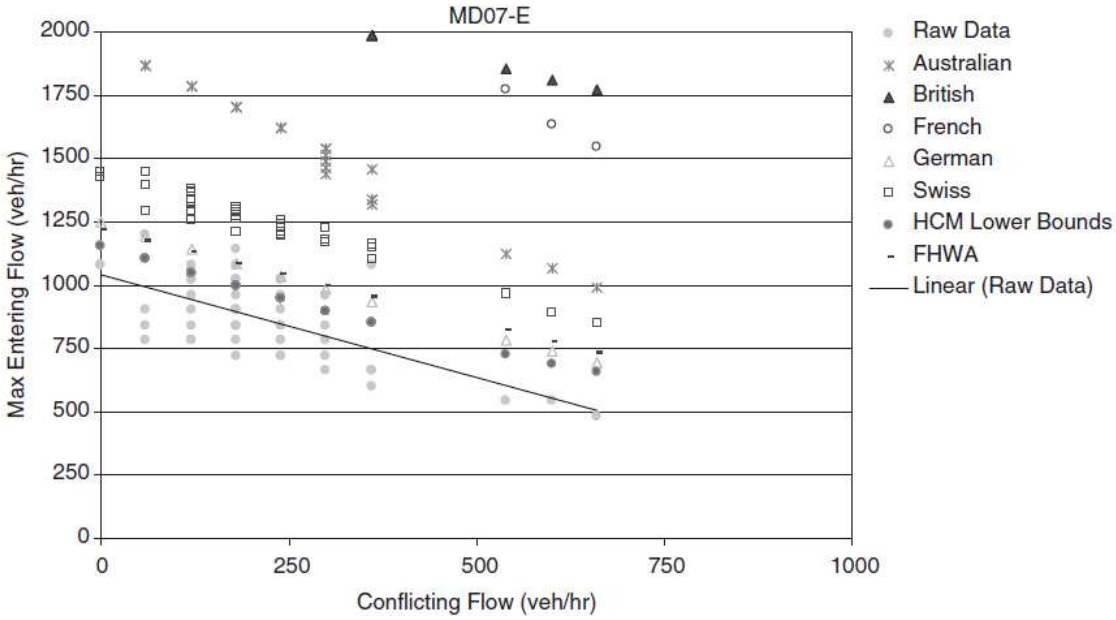

Figure 2. Existing (as of 2006) roundabout capacity equations and field data at a single-lane roundabout [1]

Given the failure of the existing models to adequately reflect US operations NCHRP project 3-65 developed recommended new capacity model for single-lane and multi-lane roundabouts. Subsequently these models are included in NCHRP Report 572, NCHRP Report 672, the second edition of FHWA's "Roundabouts: An Informational Guide" [3] as well as the 2010 edition of the Highway Capacity Manual (HCM 2010)[2]. These models are presented in the next section.

2.3 UNITED STATES ROUNDABOUT MODELS

In the capacity analysis of a roundabout each approach entry lane is modeled separately. As these models are incorporated into the HCM 2010 they are "unique in HCM 2010 in the sense that HCM models for other intersection types are by lane groups" [11]. Figure 3 shows the traffic flows at a single-lane roundabout approach. Here, $v_{ex}, v_c, and\ v_e$ represent exiting, circulating, and entering vehicle volumes, respectively.

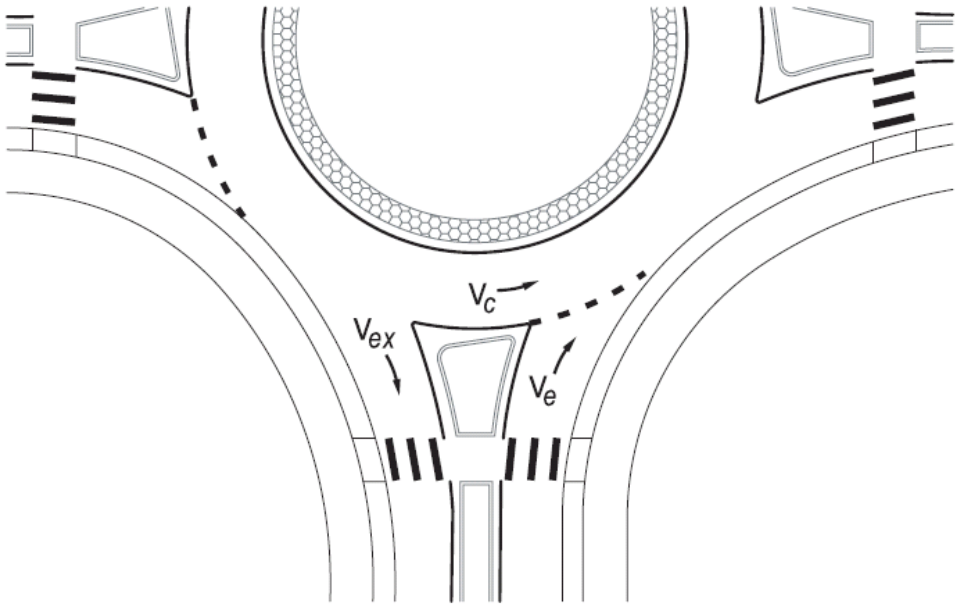

Figure 3. Circulating flow, entering flow, and exiting flow at a single-lane roundabout approach [3]

2.3.1 Capacity Equations

There are three capacity equations presented in the HCM 2010 that collectively apply to five approach and circulating lane configurations. The first capacity equation, Equation 1 below, predicts the capacity for single-lane roundabouts in passenger car equivalents per hour for conditions in which the approach and circular roadway are both single-lane [2]. Figure 4 compares the predictions from Equation 1 along with field data from NCHRP Project 3-65. NCHRP Report 572 comments that both exponential and linear regressions were considered for the model, however, the exponential equation was ultimately selected [1]. This model is also used to determine the capacity of each lane of a two lane approach where there is a single lane circular roadway [2].

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

$$c_{e,pce} = capacity, pcu/hr$$
$$v_{c,pce} = conflicting\ flow\ rate, pcu/hr$$

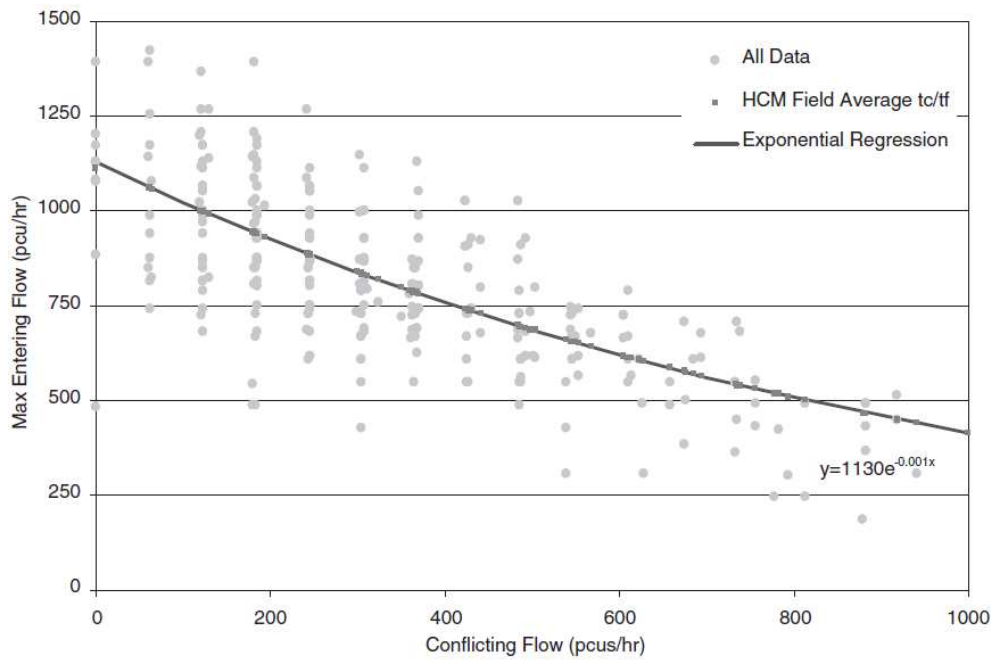

Figure 4. HCM 2010 Single-lane capacity equation with field data [1]

The HCM 2010 also provides capacity equations for multi-lane roundabouts configurations with two-lanes in the circular roadway. The HCM 2010 does not provide capacity equations for multi-lane roundabouts that have three or more circulating lanes [2]. For a roundabout approach configuration with two circulating and one approach lane, the approach lane capacity is determined according to Equation 2 [2]. Figure 5 compares the predictions of this capacity equation to the observed data from NCHRP Project 3-65 as shown in NCHRP Report 572.

$$c_{e,pce} = 1{,}130e^{(-0.7\times10^{-3})v_{c,pce}} \quad (2)$$

$$c_{e,pce} = capacity, pcu/hr$$
$$v_{c,pce} = conflicting\ flow\ rate, pcu/hr$$

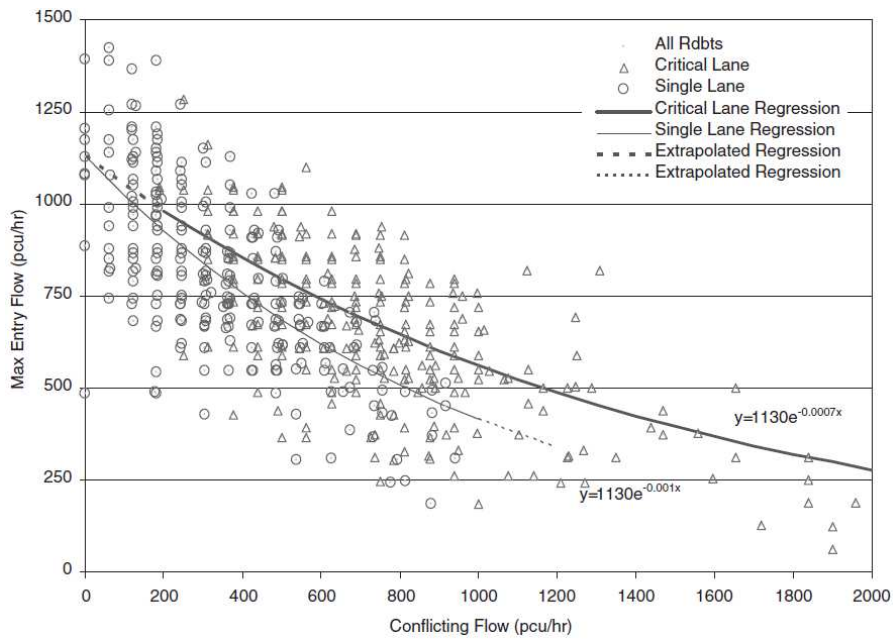

Figure 5. NCHRP capacity equations with raw data [1]

For a two-lane approach into a two-lane circulating roadway separate equations are used for each approach lane. The first is for the right lane, which according to NCHRP Report 572 is considered the critical lane as it is "the most heavily utilized lane" [1]. The right lane equation is the same as Equation 2 above. The left lane approach capacity is determined using Equation 3 below [2].

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

$$c_{e,pce} = left\ lane\ capacity, pcu/hr$$
$$v_{c,pce} = conflicting\ flow\ rate, pcu/hr$$

Slip lanes (Figure 6) are also modeled with the preceding equations. Equation 1 is applied for a single lane slip lane into an approach with a single exiting lane and Equation 2 is used for single lane slip lane into an approach with a two-lane exit. No capacity equations are given for non-yielding slip lanes, due to lack of data as this geometry is uncommon in the United States. However this geometry is mentioned and assumed to have high capacity [2].

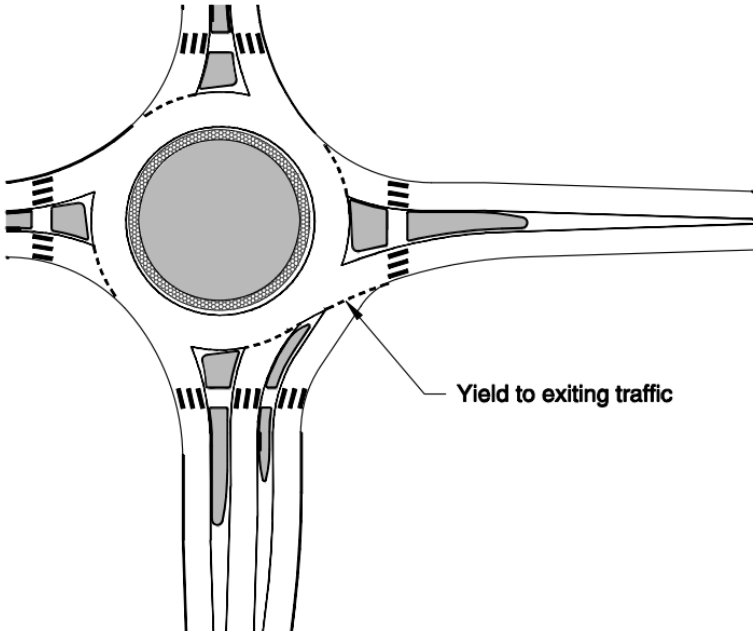

Figure 6. Example of a yielding slip lane [3]

2.3.2 General Form of Capacity Equation

As seen above, the HCM 2010 roundabout capacity equations require only the conflicting flow rate as an input. However, the general form of the equations is provided to allow for calibration of the capacity equations to local conditions [2]. These equations are designed to allow the model to stay current with the changes in driver behavior and roundabout performance [12]. The general equation takes the form of

Equation 4, where the predictive parameters in the equation are given by Equation 5 and Equation 6 [2].

$$C_{pce} = Ae^{-Bv_c} \tag{4}$$

$$A = \frac{3{,}600}{t_f} \tag{5}$$

$$B = \frac{t_c - \frac{t_f}{2}}{3{,}600} \tag{6}$$

$c_{pce} = capacity, pcu/hr$
$v_c = conflicting\ flow\ rate, pcu/hr$
$t_f = follow - up\ headway, sec$
$t_c = critical\ headway, sec.$

NCHRP Report 572 defines critical headway at a roundabout as "the minimum headway an entering driver would find acceptable" [1]. Critical headway cannot be directly measured in the field as a gap accepted by a driver may be larger than that driver's critical headway [1]. However, critical headway can be estimated based on the acceptance and rejection of gaps utilizing the maximum likelihood or similar method. Figure 7 illustrates an example of gap acceptance. In Figure 7, we assume Vehicle A and Vehicle C are circulating in the roundabout and Vehicle B enters the roundabout between Vehicle A and Vehicle C. Thus, the gap Vehicle B accepts between Vehicle A and Vehicle C is greater than or equal to the critical headway.

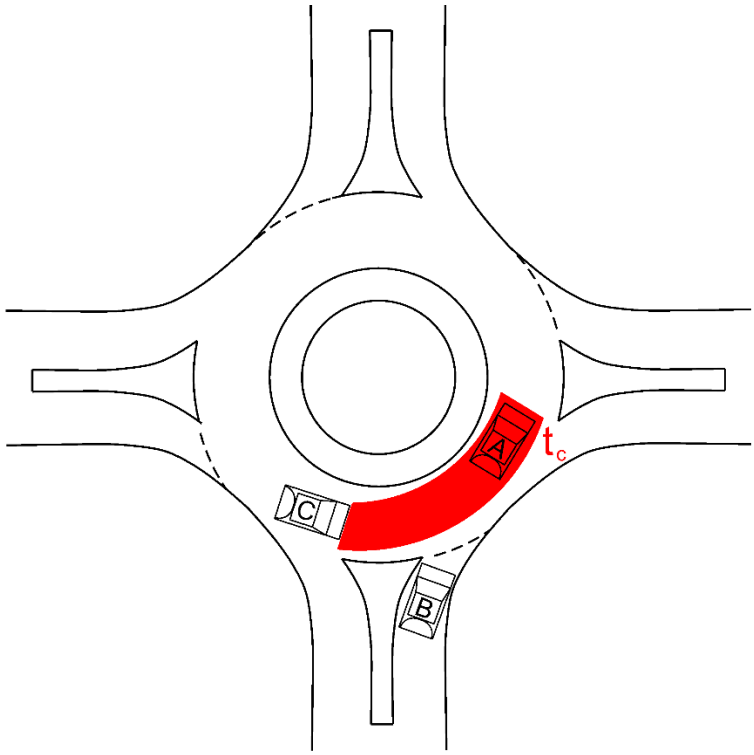

Figure 7. Critical headway example

Additionally, lags measured at the roundabout can also be used in the calculation of critical headway. A lag is the time between when a vehicle arrives at the entrance point and the next circulating vehicle. According to NCHRP Report 572, a lag is a portion of a larger gap [1].

The second major parameter, follow-up headway, is defined as "the headway maintained by two consecutive entering vehicles using the same gap in the conflicting stream" [1]. Unlike critical headway follow-up headway can be directly measured in the field. To measure follow-up headway vehicles must be queued on the roundabout approach, indicating operations at near-capacity conditions for that approach. In Figure 8 we assume that Vehicles A and B are entering the roundabout consecutively. The follow-up headway is the time between Vehicle A and Vehicle B entering the roundabout.

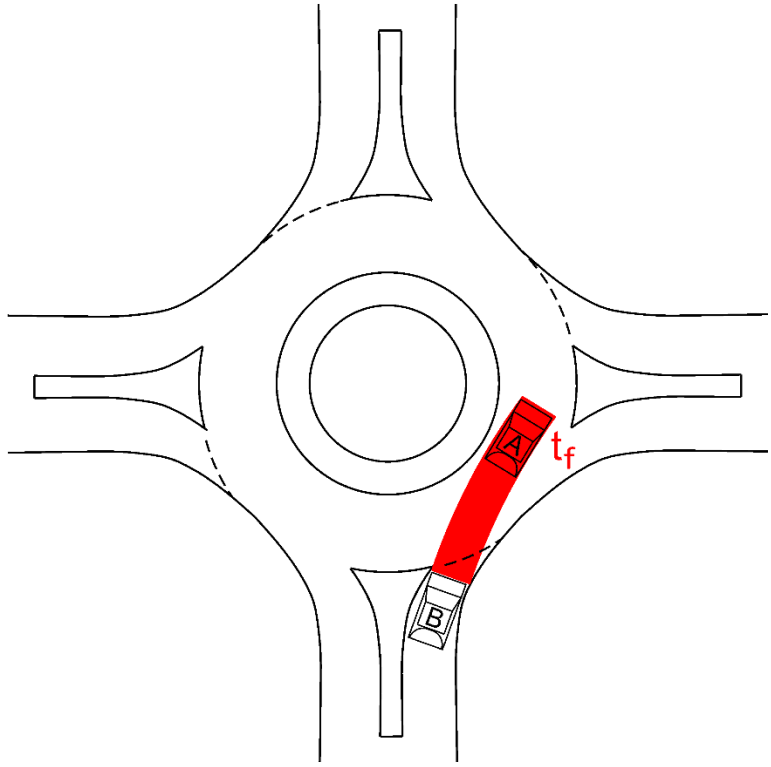

Figure 8. Follow-up headway example

2.3.3 Limitations

The HCM recognizes that these equations have limitations and, in certain situations, using other means for determining capacity may be advisable. For instance, roundabouts that have unusually high volumes of pedestrians and bicycles and use signals to accommodate these users could be modeled using other methods. Also, multi-lane roundabouts that have three or more lanes in the circulating roadway are not covered by the HCM equations and thus another analysis method would be needed to analyze a roundabout with this geometry [2].

2.4 DATA REQUIRED FOR CALIBRATION

From Equation 4, Equation 5, and Equation 6 it is seen that follow-up headway and critical headway are required to calibrate the 2010 HCM capacity equations to local driving conditions. In NCHRP project 3-65 roundabout operations were video recorded in the field and a computer-assisted method was used to extract the timestamps necessary to determine critical headway and follow-up headway. Figure 9 below from NCHRP Report 572 shows the locations from which vehicle timestamp data were extracted from the recorded videos. Points “1” and “2” were used to provide the timestamps for when an entering vehicle arrives at the yield point and when it enters the roundabout respectively. Similarly, for circulating vehicles a timestamp is collected when it either reaches point “s” at the conflict point if it remains in the roundabout or point “a” if the vehicle exits on the subject approach. The way in which these timestamps can be used to determine follow-up and critical headway is discussed in subsequent sections.

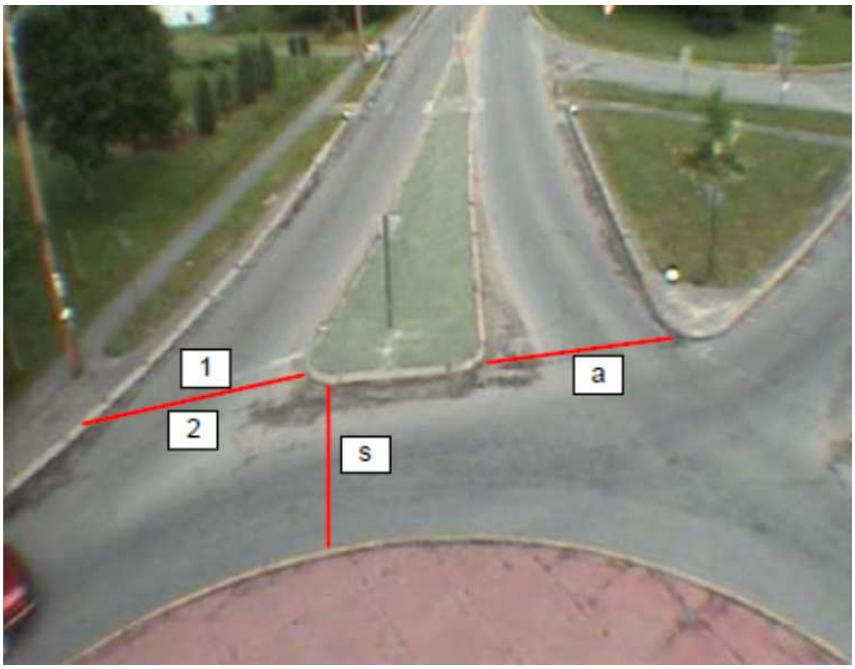

Figure 9. Physical location of timestamps [1]

2.4.1 Follow-up Headway

As stated earlier, follow-up headway can be determined directly from operations observed at the roundabout. The follow-up headway is measured by subtracting the "2" timestamp of the first vehicle from the "2" timestamp of the second vehicle. Two methods for identifying follow-up headways under capacity conditions are presented in NCHRP Report 572: 1) the "queued" method and 2) the "move-up time" method. The "queued" method requires that a standing queue be present at the approach to measure a vehicle's follow-up headway. This ensures that the measured follow-up headway reflects of capacity conditions rather than the approach arrival rate. For the determination of the average follow-up headway NCHRP Project 3-65 included follow-up headway observations that occurred only during queued conditions that lasted at least one minute for this method.

NCHRP Project 3-65 also introduced the concept of identifying follow-up headways under capacity conditions based on move-up time to allow for data collection on approaches where consistent queuing was not observed [1]. Move-up time is the time it takes for one entry vehicle to replace the prior entry vehicle at the yield point on the roundabout approach. Thus, for each follow-up headway value there is an associated move-up time value. To determine a threshold move-up time representative of near capacity conditions NCHRP Report 572 utilized the 95th percentile of all move-up times that occurred under queuing conditions that lasted at least one minute [1]. This threshold value was then applied to all follow-up data regardless of the queuing condition. If the move-up time for a follow-up headway observation was less than the threshold value it was assumed this indicated a near capacity event; therefore, the follow-up headway observation could be included in the calculation of the average follow-up headway. Appendix A provides an example of these follow-up headway calculations.

NCHRP Report 572 found an average follow-up headway value of 3.4 seconds and 3.2 seconds using the queued and move-up time methods, respectively [1]. In addition, the move-up time threshold was found to be 6 seconds. The use of move-up time expanded the number of follow-up observations by approximately 40%. The follow-up headway value used in NCHRP Report 572 and the 2010 HCM single lane roundabouts models is 3.2 seconds consistent with move-up time method.

2.4.2 Critical Headway

The critical headway is the smallest headway (or gap) in circulating traffic that an entering vehicle is willing to accept. NCHRP Report 572 presents the following three methods for determining critical headway [1]:

> *(1) inclusion of all observations of gap acceptance, including accepted lags;*
> *(2) inclusion of only observations that contain a rejected gap; and*
> *(3) inclusion of only observations where queuing was observed during the entire minute and the driver rejected a gap.*

In order to estimate the critical headway the gaps and lags an entering vehicle chooses to accept or reject are measured. A gap is measured by subtracting subsequent timestamps at line "s". A lag is "the time from the arrival of the entering vehicle at the roundabout entry to the arrival of the next conflicting vehicle" [1]. A lag is measured by subtracting the timestamp of a circulating vehicle arriving at line "s" from the timestamp of a vehicle at line "1". Lags are only measured for Method 1. In order to include lags for Method 1 "the lags have been converted to gaps using an approximate follow-up headway" [1].

NCHRP Report 572 uses a maximum likelihood method to estimate the critical headway [1]. In a previous study, Troutbeck [14] compared a maximum likelihood method to several different methods and found that the maximum likelihood method had a lower bias compared to some of the other methods such as the Ashworth method or the Ramsey and Routledge method. The maximum likelihood method is further discussed in section 3.5 of this report.

The critical headway values determined for each critical headway method are summarized in Table 1. The table shows Method 1 has a smaller critical headway value

than Method 2, implying a significant impact of including lags and not requiring a rejected gap. Also, consistent with the increasing constraints on usable data, the number of observations decreased from Method 1 to Method 2 and from Method 2 to Method 3. The final capacity equation presented in NCHRP Report 572 and the 2010 HCM used critical headway Method 2 and a critical headway value of 5.0 seconds. Appendix A provides examples calculations for each of the three critical headway methods.

Table 1. NCHRP Report 572 critical headway values for each method

NCHRP Critical Headway Method	Critical Headway Value (seconds)	Number of Observations
Method 1	4.5	11,581
Method 2	5.0	3,322
Method 3	5.1	558

2.4.3 Exiting Vehicle

As stated the line corresponding with "a" is the exit time of a vehicle on the circular roadway. However, exiting vehicles were not included in the calibration of the final capacity equations in NCHRP 572. Chapter 3 and Appendix F will discuss the incorporation of these exiting vehicles in the capacity equations.

2.5 CALIBRATION EFFORTS/CASE STUDIES

2.5.1 Caltrans

One of the first calibrations of the NCHRP 572 capacity equations was performed for the California Department of Transportation (Caltrans). Caltrans' *Roundabout Geometric Design Guidance,* which includes the calibrated capacity equations, was published in June of 2007 [15]. This study found values for critical headway and follow-up headway for single-lane roundabouts as well as for the right and left lanes of multi-lane roundabouts [15].

For the Caltrans calibration effort, traffic data were collected at seven single-lane and three multi-lane roundabouts. A minimum of two hours of video were collected at each roundabout using a tripod-mounted camera positioned to capture the traffic on the most heavily utilized leg of the roundabout. Videos were recorded at peak hours between 7:00am -9:00am, 11:30am-1:30pm, or 4:00pm-6:00pm depending on the roundabout. At several of the roundabouts four hours of data were collected and thus a total of 26 hours of traffic data were recorded [15]. Similar to the NCHRP study, critical headway and follow-up headway were found by extracting timestamp data for vehicle positions as indicated in Figure 9 [15]. Similar to the NCHRP report follow-up headway was determined directly from the field data and a Maximum Likelihood methodology was used to find the critical headway.

For single lane roundabouts the determined critical headway was 4.8 seconds while the follow-up was 2.5 seconds. For multi-lane roundabouts the critical and follow-up headways determined were: right lane - 4.7 seconds, left lane - 4.4 seconds, and

both left and right lane - 2.2 seconds [15]. The resulting calibrated capacity equations for single lane roundabouts (Equation 7), right-lane of multilane approach (Equation 8), and left lane of multilane approach (Equation 9) are given below. With slightly lower critical and follow-up headways than those found in NCHRP project 3-65 these equations result in somewhat higher capacities than those found in the 2010 HCM or NCHRP Report 572. A comparison of capacity equations from all of the case studies will be shown later in Section 4.6.

$$c = 1{,}440e^{-0.0010\times v_c} \tag{7}$$

$$c = single\ lane\ capacity, pcu/hr$$
$$v_c = conflicting\ flow\ rate, pcu/hr$$

.

$$c = 1{,}640e^{-0.0009\times v_c} \tag{8}$$

$$c = right\ lane\ capacity, pcu/hr$$
$$v_c = conflicting\ flow\ rate, pcu/hr$$

$$c = 1{,}640e^{-0.0010\times v_c} \tag{9}$$

$$c = left\ lane\ capacity, pcu/hr$$
$$v_c = conflicting\ flow\ rate, pcu/hr$$

2.5.2 City of Bend, Oregon

The City of Bend, Oregon calibrated roundabout capacity equations are presented in the *City of Bend Roundabout Operational Analysis Guidelines,* prepared for the city by Kittelson & Associates, Inc. in 2009 [16]. A critical headway of 4.1 seconds and follow-up headway of 2.7 seconds for single-lane roundabouts was determined [16] in this study. Similar to the Caltrans study, these values are lower than the NCHRP

Report 572 values (i.e. 5.0 second critical headway and 3.2. second follow-up headway) resulting in higher predicted capacities. The calibrated values resulted in calibrated single lane roundabout given as Equation 10.

$$c_{e,pce} = 1{,}130e^{(-0.0007)v_{c,pce}} \tag{10}$$

$$c_{e,pce} = capacity, pcu$$
$$v_{c,pce} = conflicting\ flow\ rate, pcu$$

2.5.3 Anchorage, Alaska

A case study to calibrate the NCHRP equation for multi-lane roundabout was performed in Alaska. This case study involved data collection at one roundabout in Anchorage, Alaska. An image of this roundabout is shown in Figure 10.

Figure 10.Roundabout in Anchorage, Alaska (Source: Google Maps, accessed 6/28/2012)

Although there are two roundabouts, one on either side of New Seward Highway, only the roundabout on the west side was used in this case study. This

roundabout includes pedestrian sidewalks, right turn slip lanes, and is the intersection of a two-way roadway with a one-way roadway [17]. The critical headway and follow-up headway were found to be 4.28 seconds and 2.58 seconds, respectively [17]. These two values yield Equation 11.

$$c_{crit} = 1{,}395e^{-0.0008v_c} \tag{11}$$

$$c_{critcal} = capacity, pcu/hr$$
$$v_c = conflicting\ flow\ rate, pcu/hr$$

It was expected that the calibrated equation would provide higher capacity estimates than the HCM 2010 model and upon comparison this was found to be the case for conflicting vehicle flows up to 2100 pcu/hr [17].

2.5.4 Wisconsin

Wisconsin has more than 200 modern roundabouts in operation [18]. The Wisconsin Traffic Operations and Safety Laboratory at the University of Wisconsin-Madison conducted a study of capacity at Wisconsin roundabouts. Four roundabouts were included in the capacity study, two single-lane and two multi-lane roundabouts. A total of five approaches were studied as two approaches were included on one of the multi-lane roundabouts [18].

For the capacity study, video data was collected using the Miovision™ camera system and HD video cameras. Cameras were placed upstream and downstream of the approach as well as one on the perimeter of the roundabout [18]. Timestamps were collected from the videos for use in determining the follow-up and critical headway. Follow-up headway was only considered under saturated conditions, which were

determined by the research team to be, “a state when at least one vehicle has been waiting behind before the leading vehicle entered the roundabout” [18]. The average follow-up headway for the two single-lane roundabouts were 2.6 seconds and 3.8 seconds. For the three studied multi-lane roundabouts approaches the follow-up headways were found to be 3.0, 2.8, and 2.4 seconds [18].

Critical headway was determined using a maximum likelihood method, similar to NCHRP Repot 572 [18]. The critical headways at the single lane roundabouts were 5.5 seconds and 4.8 seconds. For the multi-lane roundabouts the critical headways were found for the right and left lanes separately. For the right lane of the multi-lane roundabouts the critical headways were found to be 3.4 seconds, 3.8 seconds, and 4.4 seconds. For the left lane the critical headways on the studied approaches were found to be 4.1 seconds, 4.2 seconds, and 4.8 seconds [18]. Overall the values are similar to those found in NCHRP Report 572.

2.5.5 Carmel, Indiana

In addition to calibrating the 2010 HCM equations to local conditions the Carmel Indiana study also explored new capacity models [19]. The authors suggested the development of localized capacity models, rather than calibration of the HCM 2010 equations, reasoning that a different model form may better represent local conditions. According to Wie et al., “underestimating or overestimating roundabout capacity can have significant implications for decision makers” [19]. Overestimating capacity can lead to the creation of premature congestion and underestimating capacity can lead to

selection of another intersection type when a roundabout would in practice perform adequately [19].

Data were collected in one-minute periods at three roundabouts that experienced congestion or queuing in at least 15 one-minute periods per hour. Similar to the Wisconsin study, data collection was performed using the Miovision™ Video Collection Unit to record operations [19].

For this study, data extraction from the video was performed using two different methods; the Miovision™ data extraction program and manual data extraction for a subset of the data. The study references the extracted data as "unverified data", i.e. the automated data extraction, and "verified data" i.e. the manually extracted data [19]. Upon comparing the data, the automatically extracted data had a high rate of accuracy. Figure 11 shows a graphical comparison of the two data sets [19].

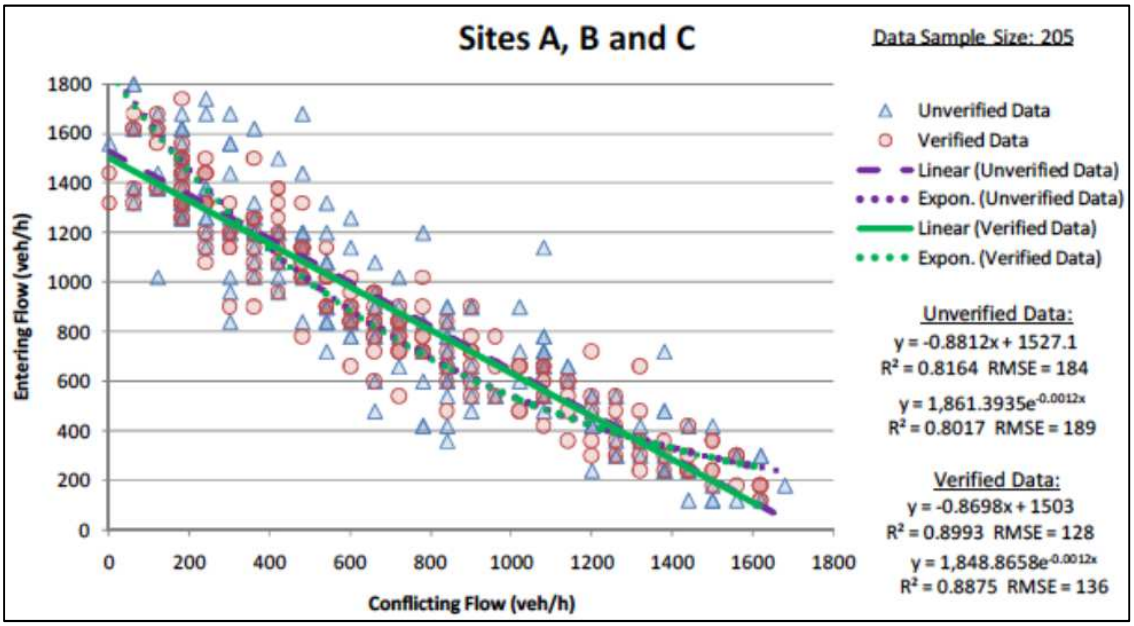

Figure 11. Comparison between verified and unverified data [5]

Similar to NCHRP Report 572 both linear and exponential regressions were applied to the data. After a comparison of the R squared values, it was determined that linear regression was a better fit for the data collected in this study. The linear capacity model was also compared with the uncalibrated (default) NCHRP model and it was found that the study's model consistently provided higher estimates of capacity. Figure 12 shows the difference between the uncalibrated NCHRP model and the model developed in this case study [19].

Critical headway and follow-up headway were also extracted from the data and found to be lower than the NCHRP values. Thus the calibrated NCHRP 572 equations also resulted in higher estimates of capacity than the uncalibrated equations.

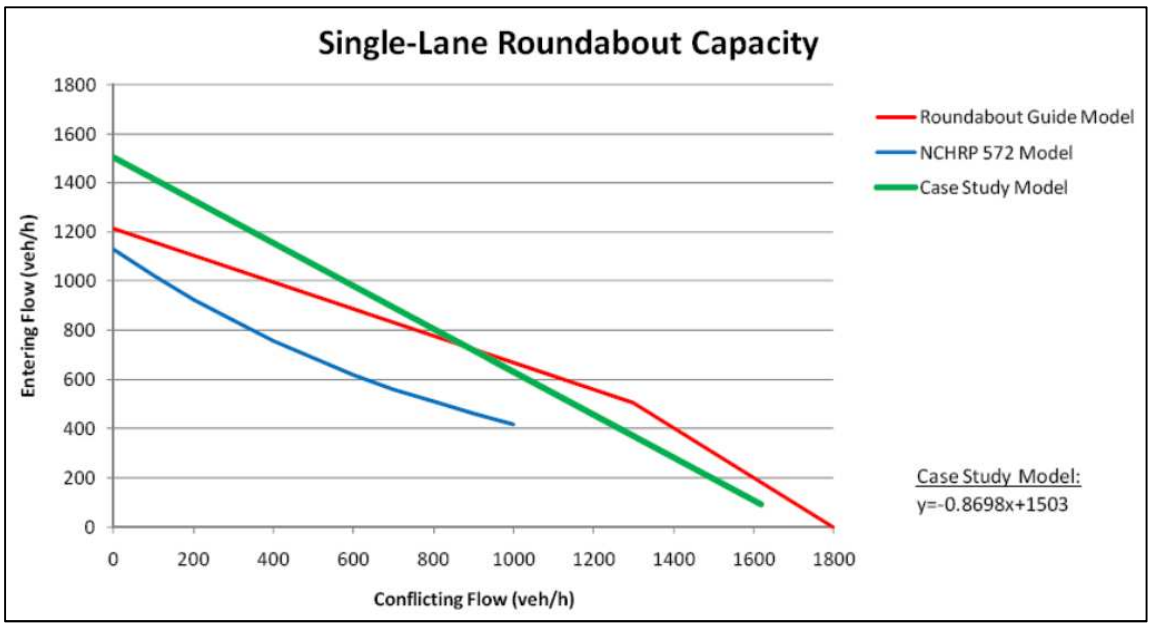

Figure 12. Comparison of capacity equations [5]

2.6 OTHER FACTORS

2.6.1 Effect of Exiting Vehicles

When an entry vehicle is waiting to enter the roundabout on an approach, the driver is examining gaps they are willing to accept between circulating vehicles. The entering driver's decision may also be impacted by the vehicles exiting the roundabout on the approach. The entering driver may hesitate to enter the roundabout until they know whether the circulating vehicle is going to exit the roundabout or continue to circulate. The current HCM 2010 model does not include the impact of these exiting vehicles in their capacity model. NCHRP Project 6-35 found that "the exiting flow does not impact all entering vehicles, and the exact extent of the influence of exiting vehicles has not been determined" [1]. However, a study performed by Mereszczak et al. [20] found that capacity predictions are improved when exiting vehicles are included as part of the capacity prediction analysis.

When exiting vehicles are included in the analysis, the following three values are impacted: conflicting flow, follow-up headway, and critical headway. For analyses including exiting vehicles, conflicting flow is the sum of the circulating flow (vehicles crossing point "s" in Figure 9) and the exiting flow (vehicles crossing point "a" in (Figure 9). For follow-up and critical headway, measurements must also be adjusted to include the effect of exiting vehicles. Mereszczak et al. found that entering vehicles will treat every vehicle, exiting or circulating, in the circulatory roadway as a circulating vehicle until that vehicle exits or that vehicle makes their intention to exit known [20]. Thus, the concept of projected travel time is used to incorporate exiting vehicles into follow-up headway and critical headway calculations. Projected travel time is the time it would take a vehicle to travel from the exiting vehicle timestamp collection location (the "a"

event) to the circulating timestamp collection location (the “s” event) as shown in Figure 13.

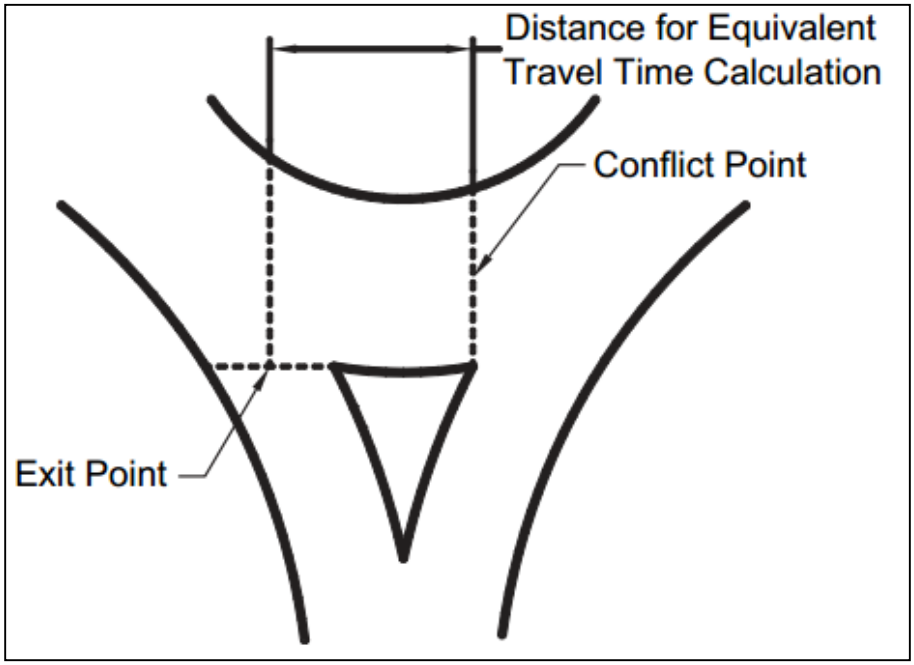

Figure 13. Projected travel time [20]

A study performed by the Wisconsin Traffic Operations and Safety (TOPS) Laboratory at the University of Wisconsin-Madison used Equation 12 to calculate gaps/lags when exiting vehicles are considered in the capacity analysis [18]. The projected travel time is added to the gap or lag only when the second circulating vehicle, T_2, is an exiting vehicle. Figure 14 provides an outline for navigating what the adjustment time should be based on the vehicle type of T_1 and T_2 (i.e. circulating, entering, or exiting). Appendix B provides a sample calculation for measuring the gap/lag when considering exiting vehicles.

$$t = T_2 - T_1 + \Delta t \quad \textbf{(12)}$$

Where:

t = Gap or lag (depends on what T_1, is), seconds

T_1 = Leading time stamp, seconds. When T_1 is the time stamp of a conflicting event or an exiting event as mentioned above, t is a gap; when T_1 is the time stamp of an arriving event, is a lag

T_2 = Time stamp or a conflicting event or an exiting event of the following circulating vehicle, seconds

Δt = Adjustment time, seconds. Δt = 0, when T_2 is the conflicting event; Δt = projected travel time, when T_2 is the exiting event.

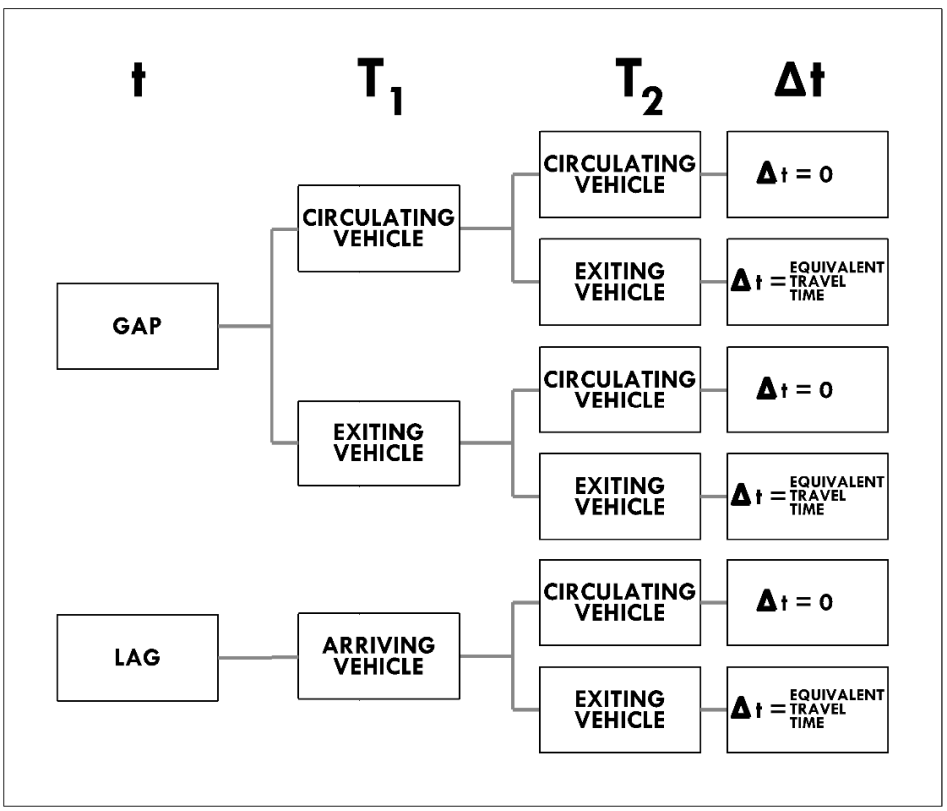

Figure 14. How to calculate adjustment time, Δt, based on T_1 and T_2 vehicle type [18].

In the Wisconsin study it was found both follow-up headway and critical headway are shorter when exiting vehicles are included [18]. Figure 15 and Figure 16 show the differences that were found in the critical headway and the follow-up headway when exiting vehicles were included.

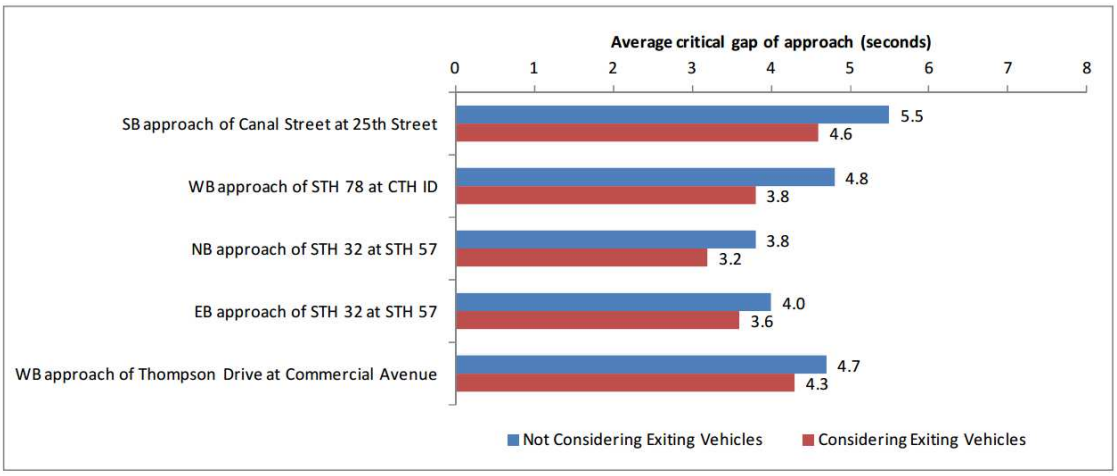

Figure 15. Effect of exiting vehicles on critical gap [18]

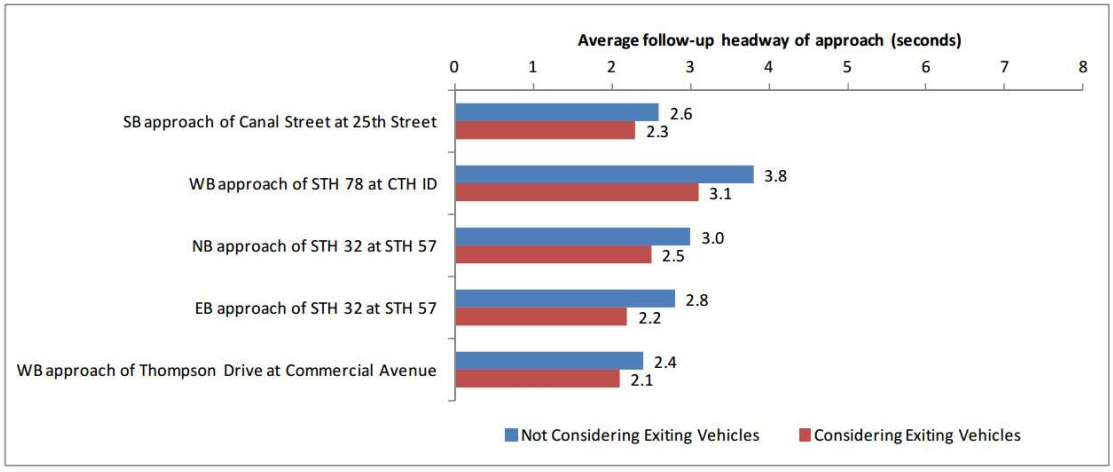

Figure 16. Effect of exiting vehicles on follow-up headway [18]

2.6.2 Effect of Trucks/Large Vehicles

The percentage of trucks and large vehicles is an additional factor that affects roundabout operations. The capacity at roundabouts with a large percentage of heavy vehicles may be affected because these vehicles have slower start-up times and different acceleration characteristics than standard passenger cars. Several studies have been completed that explored the effect of heavy vehicles on the follow-up headway and critical headway at roundabouts.

2.6.2.1 Wisconsin

In this study three vehicle types were considered: passenger cars, motorcycles, and heavy trucks. It was found that the critical headway and follow-up headway of passenger cars did not differ significantly from the headway values that were found when vehicle type was not considered. The researchers postulate that this is because motorcycles and heavy trucks were not as abundant in the traffic stream as passenger cars. It was also found that critical headway and follow-up headway are larger for trucks than for passenger cars and lower for motorcycles than for passenger cars [18].

2.6.2.2 Brattleboro, Vermont

Another study to determine the effect of heavy vehicles on roundabout capacity was conducted at a roundabout on Putney Road and Chesterfield Road in Brattleboro, Vermont. The video footage used in this study was part of the NCHRP 3-65 data collection. The authors stated that converting trucks into passenger car units (pcu) as required by the HCM 2010 roundabout capacity equations does not adequately reflect the effect that trucks have on roundabout capacity [21].

The roundabout that was used as part of this study is a single-lane roundabout with four legs. Figure 17 shows an image of this roundabout. This roundabout features a truck apron in the circulatory roadway and approximately 10-15% truck traffic.

Figure 17. Roundabout Brattleboro, Vermont (source: Google Earth™, accessed 6/28/2012).

Six hours of video data from this roundabout were used to determine the critical headway and the follow-up headway. To determine how trucks affect the follow-up headway, four different cases of follow-up headway were identified: "1) Car followed by Car (car/car), 2) Car followed by Truck (car/truck), 3) Truck followed by Car (truck/car), and 4) Truck followed by Truck (truck/truck)" [21]. Additionally, critical headway was determined separately for cars and trucks [21].

Critical headway was determined graphically by comparing the curve of accepted gaps against the curve of rejected gaps, with the intersection of these curves being the critical headway. In addition, the probability equilibrium method was used. The critical headway for trucks was found to be 5.3 seconds using both methods. The critical

headway for cars was found to be 3.8 seconds for cars using the graphical method and 3.9 seconds using the probability equilibrium method. These data shows that there is more than a second difference between car and truck headway values, clearly indicating that the presence of significant truck traffic impacts roundabout capacity [21].

In addition to critical headway, the four listed cases were evaluated for follow-up headway. The average follow-up headway for a car following a car was found to be 2.1 seconds, car following a truck was 4.1 seconds, a truck following a car was 5.3 seconds, and a truck following a truck was 8.5 seconds. However, there was only one truck following truck event was recorded. An average follow-up time was also determined from all follow-up observations involving a truck. This follow-up time was found to be 5.2 seconds, significantly longer than the 2.1 second follow-up time found for the case only involving cars [21].

The results of this study underscore the importance of calibrating these equations to local conditions. For example, an area with many trucks will have significantly less capacity in roundabouts than an area that has a very small volume of truck traffic. The authors of this study conclude by recommending that, individual values of follow-up and critical headway should be determined for cars and trucks [21].

2.7 FIELD DATA COLLECTION METHODS

The current, primary method for roundabout data collection is through the use of video cameras to record operations in the field with video post-processing (typically semi-automated) in the lab to extract the required timestamp information. These

timestamps are subsequently analyzed to determine the critical and follow-up headways. NCHRP project 3-65 was performed in the early 2000s and therefore, DVD recorders were used along with the video data collection [1]. Other, more recent studies do not mention the need of such devices, presumably because of advances in portable memory or a lack of sufficient detail in the field data description. In addition, the NCHRP study used omni-directional cameras along with standard video cameras and these cameras were placed in the central island [10]. In the Wisconsin study, several video cameras were used, and placed around the perimeter of the roundabout. These videos were then synchronized in the laboratory using a software program [18].

While the primary means for extracting data from the videos was manual observation of the video, several studies, such and the study in Wisconsin and Carmel, Indiana used Miovision™ video cameras and software. As discussed earlier Miovision™ has a software application that can automatically extract the gap data from the videos recorded with Miovision™ cameras. The study in Carmel, Indiana made use of the Miovision™ program, but also conducted manual data reduction to verify the results [19].

2.8 GDOT ROUNDABOUT ANALYSIS TOOL

GDOT provides a Roundabout Analysis Tool to assist in the planning and design of a roundabout. The user inputs the following information: vehicle volumes per hour, peak hour factor (PHF), and percent of cars, heavy vehicles, bicycles, and pedestrians [8]. Given these inputs, the tool provides feedback on the predicted operations of the proposed roundabout along with a suggested geometric design.

The GDOT tool has the ability to forecast the operations of the roundabout and provides measures of effectiveness for each approach. These operational efficiency measures are capacity, volume-to-capacity (v/c) ratio, control delay, level of service (LOS), and queue length [8]. These operational measures, specifically the v/c ratio, control delay, and queue length, assist in determining the geometric design of the roundabout. For example, if the operational measures are at an unacceptable level, the GDOT Roundabout Analysis Tool suggests adding a right turn bypass lane or making the single-lane roundabout into a multi-lane roundabout.

The tool reports two sets of results in the spreadsheet. The first set of results is labeled "HCM 2010 Model (build)" and is based on the roundabout capacity model outlined in the HCM 2010. The HCM 2010 Model (build) uses the HCM 2010 default follow-up and critical headway values, 3.2 and 5.0 seconds respectively. The second set labeled "Calibrated Model (future)" is the HCM 2010 model that was calibrated with data from Bend, Oregon and California. The calibrated model uses 2.7 and 4.1 for the follow-up and critical headway values respectively based on roundabouts in Bend, Oregon. In this instance, the calibrated results yield higher entry capacities than the non-calibrated results because the calibrated model uses lower headway values. The lower the headway values the higher the capacity.

GDOT suggests that users of the spreadsheet use the HCM 2010 Model (build) results based on the assumption that drivers in Georgia are not as familiar with roundabout as drivers in Oregon or California. The HCM 2010 Model (build) yields more conservative results than the Calibrated Model (future). GDOT suggests using the Calibrated Model (future) when roundabouts become more prominent in Georgia and Georgia drivers become more accustomed to driving in roundabouts. Table 2 provides a

summary of the follow-up and critical headway values for all of the models discussed thus far. Figure 18 displays the capacity prediction models.

Table 2. Follow-up and critical headway values for various capacity prediction models

Model	Follow-up Headway (seconds)	Critical Headway (seconds)	Exiting vehicles considered?
Bend, Oregon/ GDOT Calibrated Model (future)	2.7	4.1	No
Caltrans	2.5	4.8	No
HCM 2010 Model/GDOT HCM 2010 Model (build)	3.2	5.0	No
Wisconsin			
Canal Street at 25th St.	2.6	5.5	No
	2.3	4.6	Yes
Sth 78 at CTH ID	3.8	4.8	No
	3.1	3.8	Yes

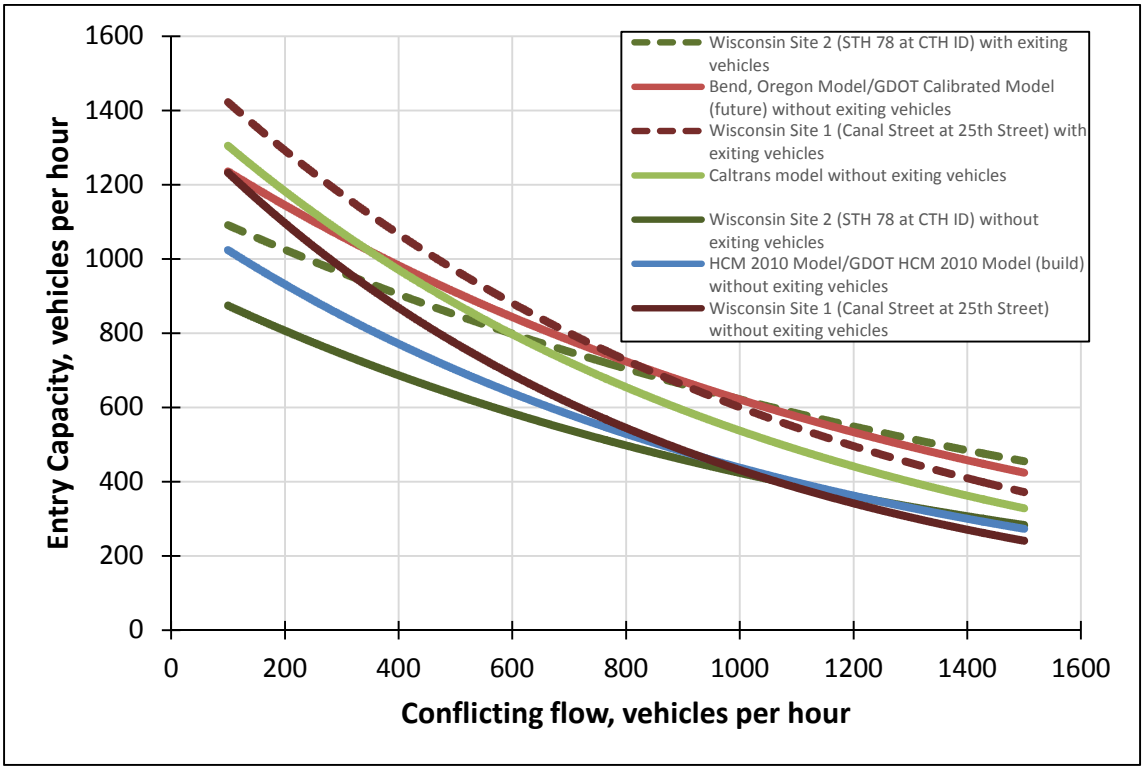

Figure 18. Comparison of entry capacity prediction models

This study seeks to develop calibrated models that can be used to supplement or to replace the use of the current uncalibrated (i.e. default) version and calibrated (future) version of the HCM 2010 model based on the California and Bend, Oregon data with models determined for Georgia conditions.

CHAPTER 3: METHODOLOGY

This section presents a detailed account of the methodology used for this study to calibrate the HCM 2010 capacity prediction equations for Georgia conditions. Similar to other studies, the first phase of this project was to create an inventory of roundabouts in Georgia from which data collection sites were selected. For the selected sites, the research team collected field video data that was subsequently processed through a semi-automated procedure using an in-house developed computer program to produce the timestamp data necessary to determine follow-up and critical headways. These extracted data from the videos were analyzed and used to determine follow-up and critical headway values appropriate for Georgia conditions and the results compared to current HCM 2010 single-lane roundabout capacity predictions. Figure 19 shows the steps that were followed to determine critical and follow-up headway values. The follow-up and critical headway values are calculated each with and without exiting vehicles. This chapter will provide a more detailed description of each of the steps in the process.

3.1 STEP 1: ROUNDABOUT SITE SELECTION

A selection process was developed in order to determine which roundabouts would be suitable sites for data collection. This selection process had two phases. The first phase was a broad sweep that identified the location and characteristics of roundabouts in Georgia and that could be used to exclude roundabouts with undesirable features. The second phase required a site visit to investigate the presence of queuing at the roundabout to determine its suitability for use in calibrating the capacity model.

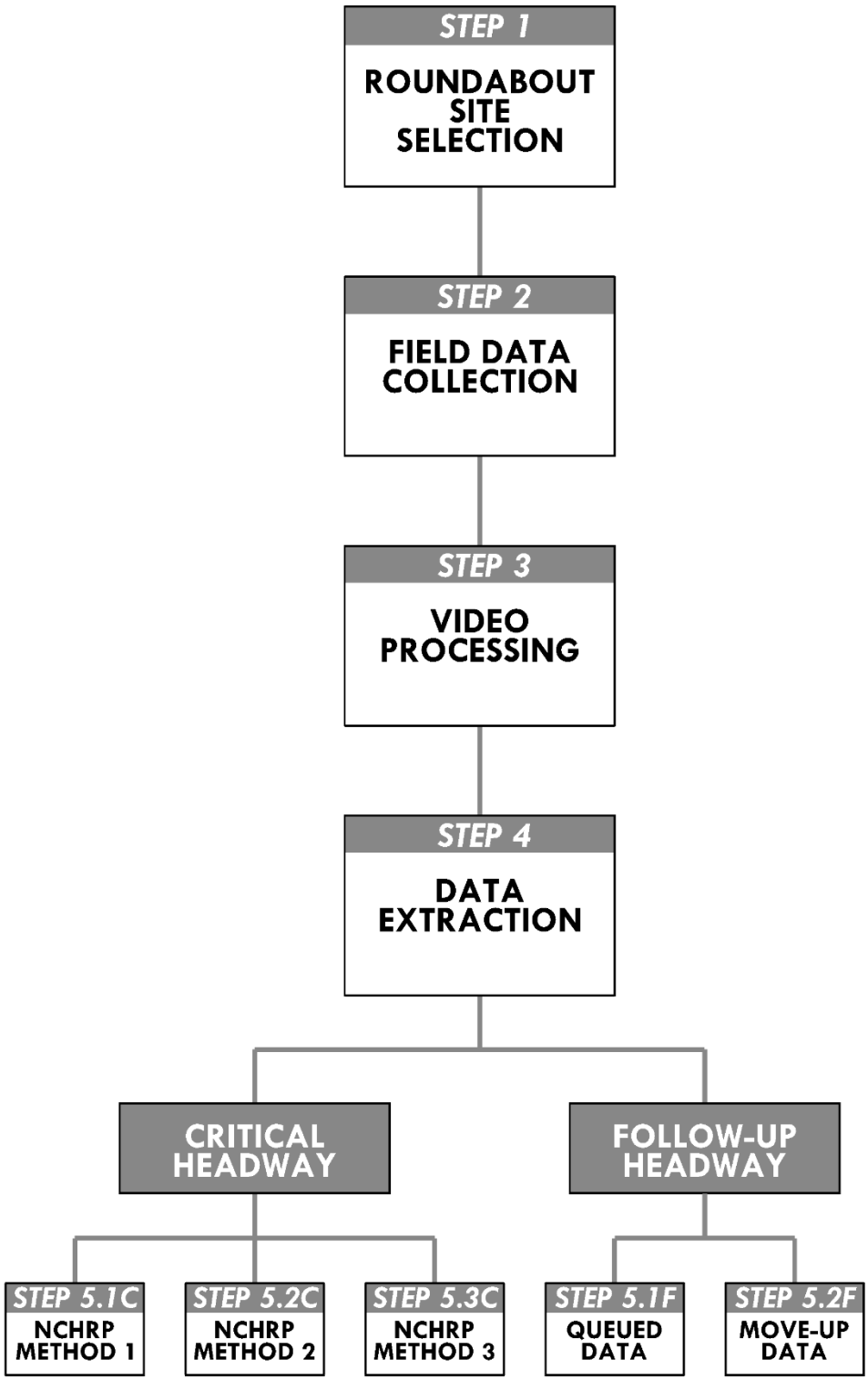

Figure 19. Steps for determining critical and follow-up headway.

3.1.1 Phase 1

The first step was to create a roundabout inventory documenting all roundabouts in Georgia. An existing list was used as a starting point and then additional roundabouts were found by searching for recently constructed roundabouts by a variety of means including searching for news accounts. The research team eventually identified over 100 roundabouts in the state of Georgia at the end of 2012. However, many of these roundabouts are low volume roundabouts located in residential areas.

Once the roundabouts in Georgia were identified, the suitability of the roundabout for collection of calibration was compared against a series of criteria:

(1) High traffic volumes

(2) Modern roundabout features

(3) Age

<u>High Traffic Volumes</u>

The GDOT State Traffic and Report Statistics (STARS) were used to identify the annual average daily traffic (AADT) of the roundabout approaches where data were available [22].

<u>Modern Roundabout Features</u>

Sites eligible for data collection had to have the modern roundabout features. Modern roundabout features include: splitter islands, truck aprons, pedestrian access, and proper signing and marking. Google Earth™ was used to inspect roundabouts for unusual geometric features [23]. For example, the circular intersection shown in Figure 20 does not have splitter islands; therefore, this site is not considered a modern roundabout. The modern roundabout features were the most important criterion when selecting roundabouts for data collection.

Figure 20. Example of a circular intersection without modern roundabout features (Source: Google Earth™, accessed September 22, 2013)

Age of Roundabout

The research team preferred the roundabout site to have been constructed at least a year prior to data collection. Newly constructed roundabouts would most likely yield highly variable driver behavior data because local drivers would still be adapting to driving in the roundabout.

3.1.2 Phase 2

The outcome of the phase one selection process was a candidate list of roundabouts with desirable characteristics. Roundabouts on the list were visited by the team to observe if there was consistent queuing on any of the approaches. The presence of queuing is necessary in order to collect data at roundabouts operating under capacity-constrained conditions.

3.2 STEP 2: FIELD DATA COLLECTION

A two person team was required for data collection. The team collected data during weekday AM peak hours, approximately 6:00 AM to 9:00 AM, and PM peak hours, approximately 4:00 PM – 7:00 PM. However, video recording was sometimes delayed or ended prematurely because of dawn or dusk conditions. The following equipment was used for data collection:

(1) 2 Panasonic HDC-TM700 video cameras
(2) 2 heavy-duty tripods with camera mounts
(3) 1 ladder 6’ – 8’
(4) 2 camera batteries

The research team filmed two roundabout approaches at each data collection site. The camera was placed on the outside of the circulating roadway and out of the view of drivers as shown in Figure 21. The camera was positioned to capture the movements of entering, circulating, and exiting traffic on the approach of interest as shown In Figure 22. In addition, the camera had to capture far enough upstream of the roundabout entry in order to determine if there is queuing on the approach. Figure 23 shows a typical view from the camera.

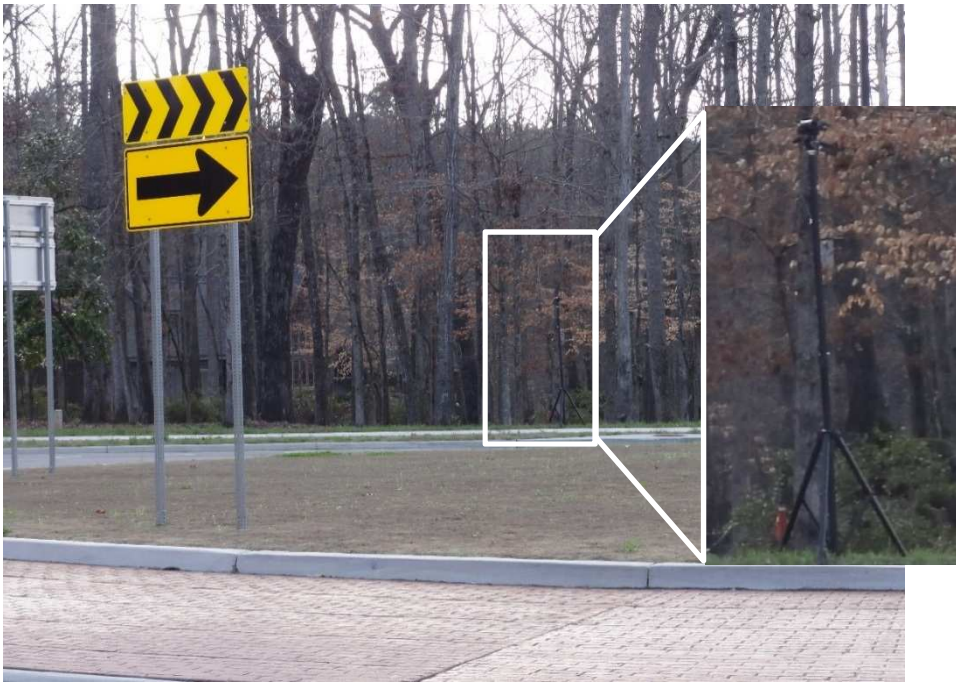

Figure 21. Camera setup for roundabout in Covington, Georgia

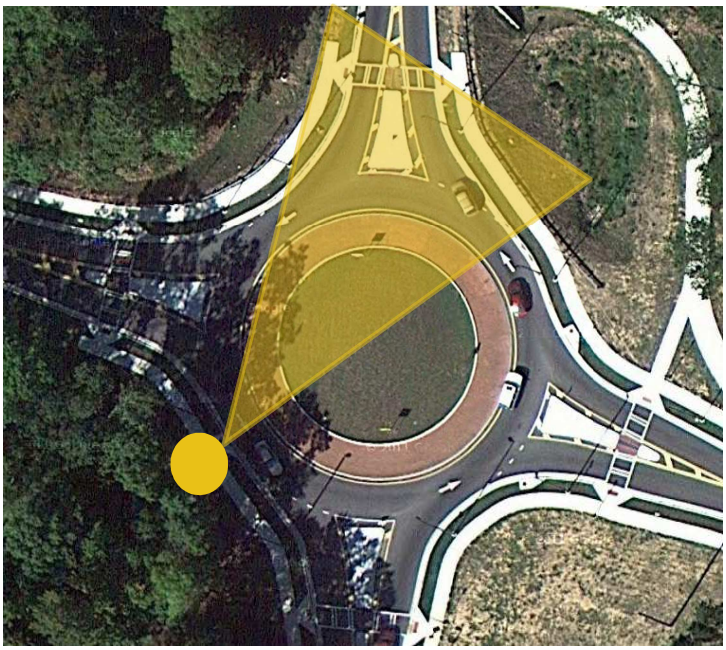

Figure 22. Camera placement for southbound approach for roundabout in Covington, Georgia

Figure 23. Camera view for southbound approach for roundabout in Covington, Georgia

3.3 STEP 3: VIDEO PROCESSING

After data collection, the timestamp data required for calculating follow-up and critical headway needed to be extracted from the videos. In order to post-process the video, the video had to be converted from mts video format to an avi file video using FFmpeg, a conversion program (Zeranoe FFmpeg 2013). The data is extracted from the videos by collecting timestamps when certain events take place. The events of interest are the following:

(1) Vehicle arrives at the entry point

(2) Vehicle enters the circular roadway

(3) Vehicle exits the roundabout

(4) Vehicle circulates in front of the approach of interest

(5) Beginning of a queue on the approach of interest

(6) Ending of a queue on the approach of interest

Events 1-4 correspond to a particular location on the roundabout, as identified previously in Figure 9. Therefore, to eliminate ambiguity and ensure repeatability of the results, lines were drawn on the video to indicate the location of where timestamps for Events 1-4 should be collected. The lines were drawn on the video using AVS Video Editor by Online Media Technologies Ltd. [25]. The lines were drawn according to the example provided in the NCHRP Web-Only Document 94 shown in the Literature Review section of this report [4]. Figure 24 displays the location of the lines on the roundabout video. Each of the lines corresponds to an event which is summarized in Table 3.

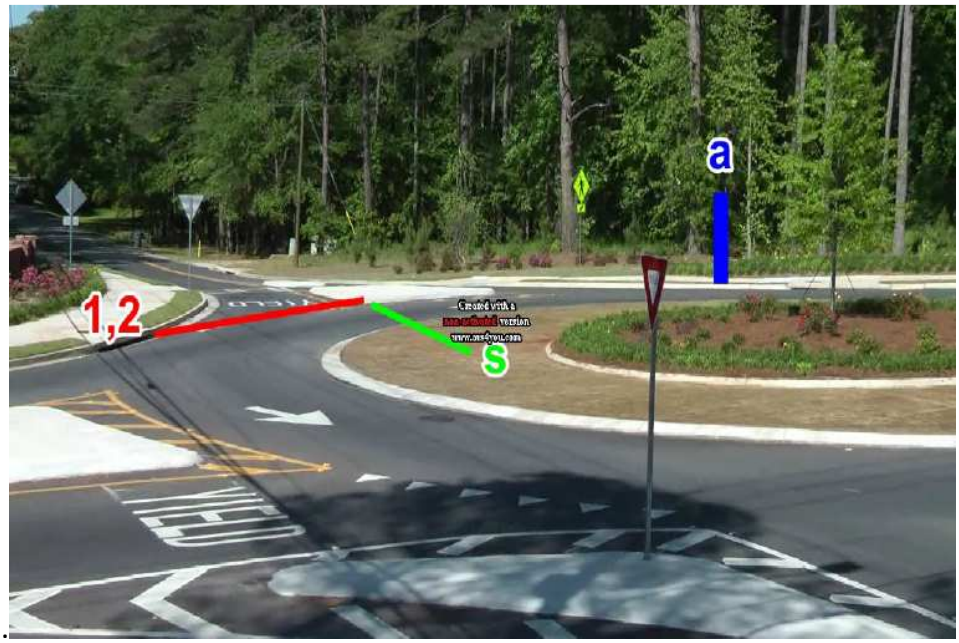

Figure 24. Location of timestamp data collection on southbound approach for roundabout in Fayetteville, Georgia

Table 3. Summary of keystrokes

Keystroke	Event
1	Vehicle arrives at the entry point
2	Vehicle enters the roundabout
a	Vehicle exits the roundabout
s	Vehicle circulates in front of the approach of interest
x	Beginning of queue on the approach
z	End of queue on the approach
q	Errors in the data collection file

The team developed a Java program that collected timestamps corresponding to the events in Table 3 [26]. Undergraduate research assistants (URAs) would review the roundabout video at real-time speed and enter the correct keyboard keystroke when an event occurred. Videos ranged in length from one to three hours.

The methodology described in the NCHRP Report 572 collects all keystrokes in a single pass through the video. After preliminary data collection, the research team found it very difficult to accurately record all the events of the vehicles in the roundabout as they occur in real-time in one pass through the video. Therefore, because collecting all data in one pass through the video yielded inaccurate results, the research team required that data collection occur in three passes through the same video. Thus, URAs would watch each video three times, collecting different keystrokes each viewing. The first pass through the video keystrokes “1” and “2” were collected. Keystrokes “a” and “s” keystrokes were collected in the second pass. Finally, the third pass through the video the keystrokes “x” and “z” were collected. The keystroke “q” was entered to denote a mistake during the data collection process. For example, if the URA identified a circulating vehicle as an exiting vehicle, the URA would immediately press “q” after making an incorrect keystroke. The mistake would later be resolved. The interface of the program is shown in Figure 25. Instructions for the data program provided to the URAs can be found in Appendix C.

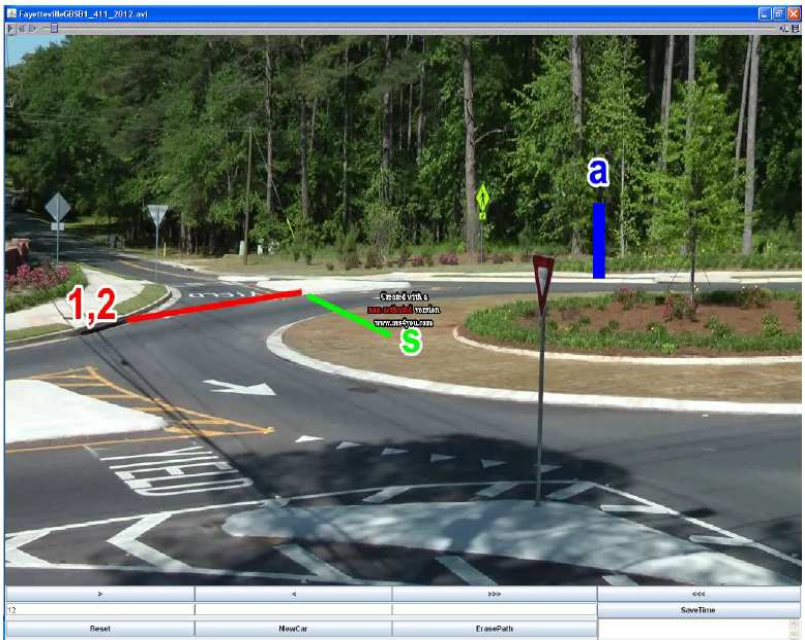

Figure 25. Interface of program used to collect timestamps

3.4 STEP 4: DATA EXTRACTION

After each pass through the video, the program would write the timestamps directly to a comma separated values (CSV) text file. After the three video reviews were completed an in-house developed Microsoft Visual Basic computer program was used to merge the three CSV files and sort the timestamps in order of smallest timestamp to largest timestamp. The merged and sorted keystroke file served as a log of all vehicle activity in the roundabout in the order the events occurred. Once this log was created the program calculated the following values:

(1) Gap and lag data

(2) Queuing periods

(3) Move-up time

The values above are used to calculate follow-up and critical headway. The purpose of this section is to describe what data was extracted from the video processing output files and how the above values were calculated.

3.4.1 Gap and Lag Data

Accepted/Rejected Gap Data

As discussed in the previous chapter a gap is the time measured between two consecutive conflicting circulating vehicles in a roundabout at some reference point [1]. The gap can be accepted or rejected by an entering vehicle waiting at the yield point. The following event sequence defines an accepted gap: 1) the first vehicle circulates ("s" event), 2) entry vehicle enters the roundabout ("2" event), 3) the second vehicle circulates ("s" event). In other words, the entry vehicle accepts a gap when the entry vehicle enters the roundabout in-between the two circulating vehicles. If the entry vehicle does not enter the roundabout between the two circulating vehicles, the entry vehicle rejects the gap. The accepted/rejected gap is measured by finding the time between the first and second circulating vehicles at the line "s" location. In all cases the entering vehicle must be waiting at the yield point, i.e. the "1" event has been recorded, prior to the first circulating vehicle passing the approach.

If exiting vehicles are included in the analysis, an accepted gap is when the following event sequence occurs: 1) the first vehicle circulates or exits ("s" or "a" event), 2) entry vehicle enters the roundabout ("2" event), 3) the second vehicle circulates or exits ("s" or "a" event). If the entry vehicle does not enter the roundabout between the two conflicting vehicles, the entry vehicle rejects the gap. The accepted/rejected gap is measured by finding the time between the first and second conflicting (circulating or exiting) vehicles at the line "s" or "a" location. The concept of projected travel time as described previously in the Wisconsin study is used when the second vehicle of the gap

is an exiting vehicle in order to project the timestamp of the exiting vehicle forward to the “s” line.

The projected travel time (Δt, Figure 13) was measured in a separate pass through each roundabout site video. Projected travel time was measured using the timestamp of a circulating vehicle passing the “a” and “s” locations. The projected travel time was found by subtracting the “a” timestamp from the “s” timestamp. For each roundabout approach site, twenty-five observations of project travel time were extracted and average to determine the projected travel time for that approach.

Rejected Lag Data

The NCHRP Report 572 defines a lag as “the time from the arrival of the entering vehicle at the roundabout entry to the arrival of the next conflicting vehicle” [1]. A rejected lag is when the following event sequence occurs: 1) entry vehicle arrives on the approach (“1” event) and 2) a vehicle circulates (“s” event). If exiting vehicles are included in the analysis, a rejected lag is when the following event sequence occurs: 1) entry vehicle arrives on the approach (“1” event) and 2) a vehicle exits (“s” or “a” event). Rejected lag is calculated by subtracting the arriving vehicle timestamp (the “1” event) from the conflicting event timestamp (“s” or “a” event). The equivalent travel time is applied to the “a” timestamp if the second vehicle is an exiting vehicle. Rejected lag data is used when calculating critical headway according to Method 1. Based on a meeting with Kittelson & Associates, Inc. on June 20, 2013 it was determined that Method 1 does not utilize accepted lags.

3.4.2 Queuing Periods

Queuing periods are found to define when there are queuing conditions on the roundabout approach. These periods are based on observation. When queuing is

present on the approach, the roundabout approach is assumed to be operating at or near capacity constrained conditions. Therefore, data corresponding to events (i.e. entering, circulating, and exiting vehicle data) that occur under capacity constrained conditions is important to identify for the calculation of critical and follow-up headway. The purpose of defining queuing periods is to indicate data that will be utilized for the follow-up and critical headway determination.

A queuing period is bounded by the "x" event and the "z" event as recorded by the data collection personnel. The developed Microsoft Visual Basic® program extracts the timestamps of the "x" and "z" keystrokes to determine if the queuing period is at least a minute long. The NCHRP Report 572's critical headway Method 3 and follow-up headway queued data method requires headway observations to take place during queuing periods lasting at least one minute [1]. The length of the queuing period is the difference in the timestamps of when the queue began ("x" event) and the time of when the queue ended (the "z" event). Queuing periods lasting at least a minute are used to indicate acceptable ranges of data for calculating critical and follow-up headway. In the analysis it was not required that queuing periods began or ended on an integer minute.

3.4.3 Move-Up Time

Move-up time is the amount of time an entry vehicle requires to replace the prior entry vehicle at the roundabout approach. The move-up time is the difference between the first entry vehicle leaving the approach ("2" event) and the second entry vehicle arriving at the approach ("1" event). As discussed in Section 2.4.1 move-up time provides an alternative means to define queued conditions. Through the use of move-up time the number of follow-up headway observations may be significantly expanded over queuing conditions identified based solely on observations

3.5 STEP 5C: CRITICAL HEADWAY

NCHRP Report 572 defines the critical headway as “the minimum headway an entering driver would find acceptable” [1]. Since critical headway is the minimum headway, it is assumed that any observed gap a driver accepts will be larger than, or equal to, the critical headway. Therefore, the critical headway cannot be directly observed in the field. The critical headway is estimated based on lag and gap acceptance and rejection. Once the gaps and lags have been identified a Maximum Likelihood Method is used to find the critical headway value.

The Maximum Likelihood Method was used to perform a logistic regression on the accepted/rejected gaps and rejected lags. The likelihood function represents the probability of gap acceptance as a function of gap time. In order to perform a logistic regression, the data must represent a dichotomy (e.g. 1 and 0 or true and false) . In this analysis accepted gaps were assigned the value of “true” or 1 indicating a successful gap acceptance. Rejected gaps and rejected lags were assigned a value of “false” or zero. The one and zero values were placed in a column labeled “Success” in the gap/lag data CSV file. These results were subsequently analyzed to determine the critical (logistical inflection) gap time through the Maximum Likelihood Method [27]. For this study, the Maximum Likelihood Method was implemented using the statistical software package “R” version 2.4.11. Figure 26 shows a graph of the gap/lag data with assigned one and zero values.

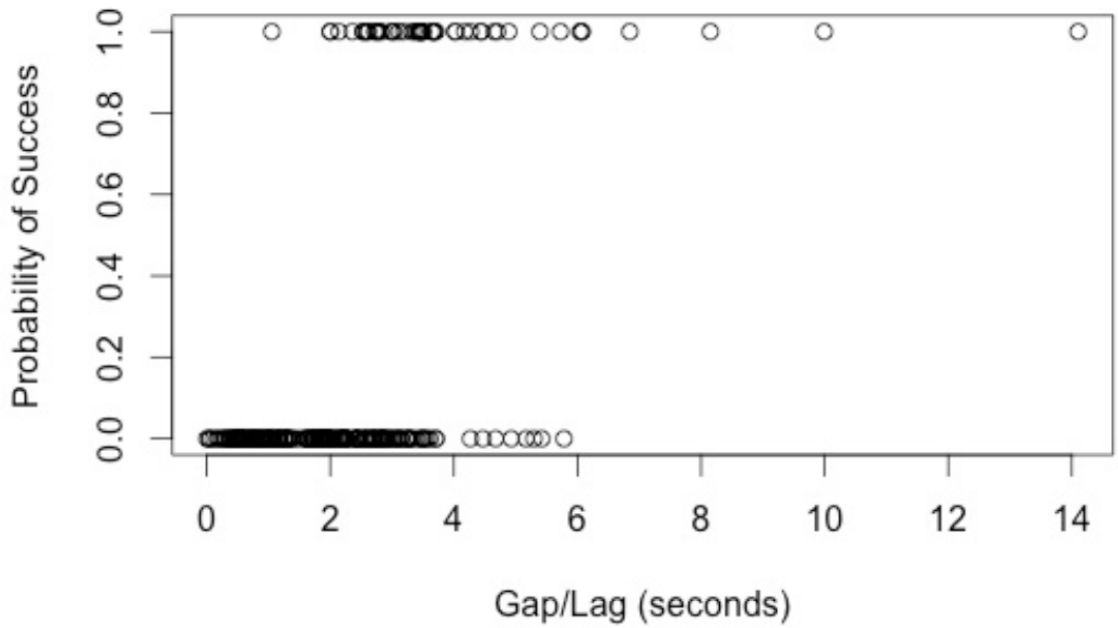

Figure 26. Gap and lag data plotted as one and zero values in R

The maximum likelihood for the critical headway is found at the inflection point of the logistic curve. This inflection point represents where the second derivative of the logistic equation is equal to zero. The logistic curve equation is found in R which has the form of Equation 13. The equation is shown below in Figure 27 displays the logistic regression with the inflection point. The inflection point is the critical headway value.

$$E\ (Y_i|X_i) = \frac{e^{\beta_0+\beta_1 X_i}}{1+e^{\beta_0+\beta_1 X_i}} \quad \textbf{(13)}$$

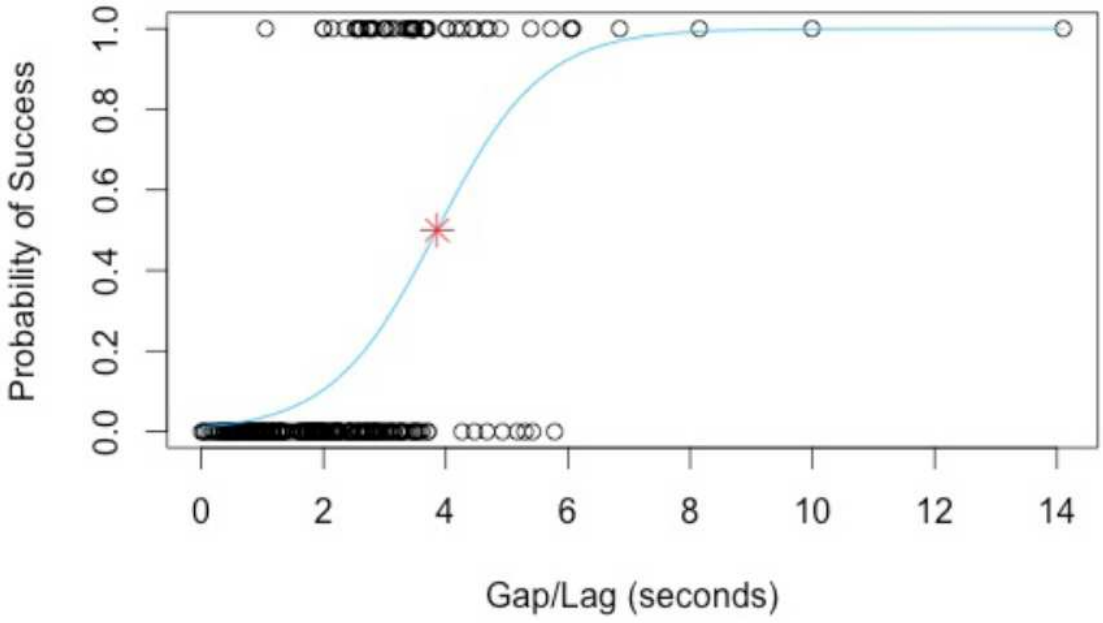

Figure 27. Logistic regression with inflection point

As mentioned in the Literature Review section of this report, the NCHRP Report 572 presents three methods for calculating critical headway. This study found critical headway values for each of the three NCHRP Report 572 methods using two different data sets: 1) with exiting vehicles and 2) without exiting vehicles. Each method is discussed in the sections below.

3.5.1 Step 5.1C: NCHRP Report 572 Critical Headway Method 1

NCHRP Report 572 defines the first critical headway method as the "inclusion of all observations of gap acceptance, including accepted lags" [1]. However, on June 20, 2013, the research team met with Kittelson & Associates, Inc. who informed the team that NCHRP Report 572's Method 1 for calculating critical headway should read as follows: inclusion of all observations of gap acceptance, including rejected lags. For this method, the data required to find critical headway is the accepted/rejected gaps and

rejected lags. Once the gaps and lags have been identified, the Maximum Likelihood Method is used to determine the critical headway value.

3.5.2 Step 5.2C: NCHRP Report 572 Critical Headway Method 2

NCHRP Report 572 defines the second critical headway method as the "inclusion of only observations that contain a rejected gap" [1]. For this method, the data required to find critical headway are the accepted/rejected gaps. However, for an accepted gap to be included in the calculation of critical headway the entry vehicle must reject at least one gap before it accepts a gap Once the gaps have been identified, the Maximum Likelihood Method is used to determine the critical headway value.

3.5.3 Step 5.3C: NCHRP Report 572 Critical Headway Method 3

NCHRP Report 572 defines the third critical headway method as the "inclusion of only observations where queuing was observed during the entire minute and the driver rejected a gap" [1]. For this method, the data required to find critical headway are the accepted/rejected gaps and the queuing periods. For an accepted gap to be included in the calculation of critical headway the entry vehicle must reject at least one gap and the accepted/rejected gaps must occur under queuing conditions of at least one minute. Once the gaps have been identified, the Maximum Likelihood Method is used to determine the critical headway value.

3.6 STEP 5F: FOLLOW-UP HEADWAY

The NCHRP Report 572 defines follow-up headway as: "the headway maintained by two consecutive entering vehicles using the same gap in the conflicting stream. The entering vehicles must be in a queue" [1]. Follow-up headway is the time difference of

first entry vehicle entering the roundabout ("2" event) and the second entry vehicle entering the roundabout ("2" event). NCHRP Report 572 identifies two methods for determining follow-up headway: 1) Queued Data Method and 2) Move-up Data Method.

3.6.1 Step 5.1F: Queued Data Method for Follow-up Headway

The Queued Data Method uses only follow-up observations that were observed during queuing conditions that were at least one minute long. A potential drawback of this method is a reduction in the number of follow-up headway observations where roundabout sites are not consistently under capacity constrained conditions.

3.6.2 Step 5.2F: Move-up Time Data Method for Follow-up Headway

The Move-up Time Data Method increases the number of follow-up headway observations by using a move-up time threshold. For every follow-up time there is a corresponding move-up time. The Move-up Time Data Method sets a threshold based on the 95th percentile move-up time for all follow-up headway observations, for all roundabout sites, during queuing conditions. Then, for all follow-up headway data, regardless of queuing, where the associated move-up time is less than or equal to the move-up time threshold the follow-up time is included in the calculation of the average follow-up headway. This expands they number of included follow-up headways as the queuing condition constraint is largely removed.

3.7 MODEL CALIBRATION

For each roundabout approach studied the average critical and follow-up headways were determined based on the methods discussed in the preceding sections. These values were used to determine the weighted average critical and follow-up

headway values across all roundabout approaches. The weighted average is determined according to the number of observation on each approach. For example, Equation 14 is the critical headway weighted average. The critical and follow-up headway weighted averages are then used in the HCM 2010 single-lane roundabout capacity equations, i.e. Equations 8, 9, and 10 given earlier.. Once calibrated, the only input required by the analyst is the conflicting vehicle volume. If the analysis is considering exiting vehicles then the conflicting vehicle volume will include the both the circulating and exiting vehicle volume.

$$\mathrm{t_c} = \sum \left[\left[t_{c_{ALP01-SB}} \times \left(\frac{n_{ALP01-SB}}{N} \right) \right] + \left[t_{c_{COV01-SB}} \times \left(\frac{n_{COV01-SB}}{N} \right) \right] + \left[t_{c_{COV01-NB}} \times \left(\frac{n_{COV01-NB}}{N} \right) \right] + \cdots \right] \qquad (14)$$

3.8 QUALITY ASSURANCE/QUALITY CONTROL

The Quality Assurance/Quality Control (QA/QC) process is important to ensure that the data reduction process can be duplicated and similar results are obtained regardless of the analyst. Two undergraduate research assistants (URA) were assigned to each roundabout video selected for data reduction. The primary URA records data for the entire video while the QA/QC URA records data for a randomly selected thirty minute subset of the video. Full length videos range in length from one to three hours. As mentioned in Step 3: Video Processing, the data collection procedure requires three passes through the video to collect the keystrokes.

A comma separated values text file (CSV) is generated for each pass through the video. Once all three passes are complete the three generated CSV files are merged and timestamps are sorted lowest to highest. The merged files of the primary and QA/QC URAs are compared to verify similarity of results. There are two types of values that are compared: 1) the timestamps and 2) the headway values.

For each keystroke type collected (i.e. “1”, “2”, “a”, and “s”), the primary and QA/QC URAs’ timestamps are compared to determine the average difference for each keystroke type. For example, suppose the average difference between the primary and URA timestamps for the “s” event is being calculated. All of the primary URA’s timestamps for keystroke “s” are placed in one column of an Excel file. In another column are the QA/QC URA’s timestamps for keystroke “s”. In order to find the average difference for keystroke “s”, the number of observations must be equal. The number of observations for each URA was checked to ensure they were equal. An unequal number of observations is indicative that one of the following scenarios occurred: 1) one URA missed an event or 2) one URA accidentally pressed a keystroke when an event did not occur. In the event that there are an unequal number of observations, the researcher performing the QA/QC analysis must watch the roundabout video to determine which of the URAs’ data sets is correct. Once the flawed data set was identified, a third URA was assigned to repeat either the primary or the QA/QC data collection, whichever data set was found to be incorrect.

Once it has been verified that both the primary and QA/QC URA have the same number of observations, the average difference can be calculated. For each occurrence of the “s” event, the difference between the timestamps corresponding to the “s” event is calculated. The average of the differences was found. This study determined a threshold of 0.2 seconds to be acceptable for the average difference. Therefore, if the average difference was greater than 0.2 seconds the researcher performing the QA/QC analysis would have to determine which data set was more accurate when compared to the video. The less accurate data was discarded and replaced with new data collected by a third URA.

After the average difference in timestamps was compared, a second check was performed. The second check compared the follow-up and critical headway values. Follow-up and critical headway values were calculated using the above methodology for each of the primary and QA/QC URA thirty minute data sets. The acceptable difference in headway values was 0.2 seconds. However, during this check the research team found that in some instances the order of the keystrokes of the two URAs were not the same. Review of the data has shown that as all timestamps are not collected in one pass through the video it is possible that when the keystrokes are merged and sorted the order of the keystrokes could be different than the actual order of events based on differences in the URAs reaction time to the different events and selected keys. The URAs would have the same number of keystrokes for each type of event (i.e. both undergraduate's sample data contains the same number of exiting vehicles, circulating vehicles, etc.) which indicates both users accurately identified all events. However, when the .csv files are merged the outcome produces a different order of keystrokes. A different order in events would create different accepted/rejected gaps and lags.

The most prominent discrepancy between the two URAs is when a circulating vehicle arrives at the "s" location and an entering arrives at the "1" location at approximately the same time. One URA would indicate the circulating vehicle arrived first while the second URA would indicate the entry vehicle arrived first. In this instance, a lag would be measured for the second URA and not for the first URA. Multiple instances of measuring different accepted/rejected gaps and lags could affect the final critical headway values. Therefore, observations where a circulating vehicle and arriving entry vehicle occur in less than 0.1 seconds of each other were excluded in the thirty minute comparison analysis. By eliminating this scenario from the URAs thirty minute data sets, the remaining data in the primary and QA/QC URA data sets would have the

same order of events. The same order of events meant the same accepted/rejected gaps and lags were being measured and allowed for a more meaningful comparison between URA collection efforts.

CHAPTER 4: RESULTS

4.1 DATA COLLECTION SITES

Following the recommendation of NCHRP 572, this chapter focuses on capacity calibration without the inclusion of exiting vehicles. This is also consistent with the HCM 2010 roundabout capacity equations and GDOT analysis tool. However, Appendix F provides the calibration including exiting vehicles for comparative purposes and section 4.6 comparing this study with previous effort will also incorporate exiting vehicle results.

Table 4 provides information regarding the roundabout data collection sites. A GDOT district map with the locations of all the data collection sites is found in Appendix D. Appendix E contains summary sheets for the field and video post-processing extracted data for each site.

Table 4. Roundabout Data Collection Sites

Site	City	Lane Configuration			Collection Date	Approach	Time Duration (hr)	Site ID.
		Circulating Lanes	Legs	Slip Lanes				
Douglas Rd./Southlake Dr./Leeward Walk Cir.	Alpharetta	1	4	0	11/13/2012	SB	1:57	ALP01-SB
Turner Lake Rd. SW/Clark St. SW	Covington	1	4	0	3/1/2012	SB	2:33	COV01-SB
						NB	2:24	COV01-NB
					5/24/2012	WB	2:25	COV01-WB
						EB	2:11	COV01-EB
Warm Springs Rd./Blackmon Rd	Columbus	1	4	0	11/2/2012	SEB	1:39	COL01-SEB
						SWB	2:07	COL01-SWB
SR 166 (Duncan Memorial Hwy)/SR 5 (Bill Arp Rd.)	Douglasville	1	4	2	5/14/2012	EB	2:17	DOU01-EB
						WB	2:10	DOU01-WB
					11/1/2012	SB	1:27	DOU01-SB
N. Decatur Rd./Oxford Rd NE.	Atlanta	1	5	1	10/19/2012	SEB	1:49	EMO01-SEB
Grady Ave./Beauregard Blvd.	Fayetteville	1	4	1	4/11/2012	EB	2:04	FAY01-EB
						SB	2:02	FAY01-SB
					10/23/2012	NB	2:19	FAY01-NB
McClure Bridge Rd./W. Lawrenceville St./Irvindale Rd. NW	Duluth	1	3	0	6/1/2012	EB	1:49	DUL01-EB
N. Main St./Memorial Dr.	Hinesville	1	4	0	7/27/2012	WB	1:11	HIN01-WB
						SB	1:39	HIN01-SB
Holly Springs Rd./Davis Rd.	Marietta	1	4	0	10/11/2012	EB	1:58	HOL01-EB
						NB	1:52	HOL01-NB
Villa Rica/Sandtown	Marietta	1	4	0	3/27/2012	SWB	1:52	VIL01-SWB
E. Broad St./Greison Tr./E. Newnan Rd.	Newnan	1	4	0	10/25/2012	EB	1:55	NEW01-EB
						WB	1:52	NEW01-WB
Grimes Bridge Rd./Norcross St./Warsaw Rd./Melody Ln.	Roswell	1	5	2	5/15/2012	EB	2:06	ROS01-EB
						SWB	2:06	ROS01-SWB
					10/26/2012	EB	2:00	ROS02-EB
						SWB	2:00	ROS02-SWB
Lawrence Rd./Frederica Rd.	St. Simons	1	3	0	7/28/2012	WB	2:52	STS01-WB
						EB	2:08	STS01-EB
						Total	**56:44:00**	

4.2 CRITICAL HEADWAY

Critical headway values for each roundabout site are determined using each of the three NCHRP critical headway methods.

Table 5 displays the critical headway values (excluding exiting vehicles) for each of the three NCHRP critical headway Methods. The Georgia-specific critical headway values for methods 1, 2, and 3 are 5.503, 4.747, and 4.922 seconds respectively.

Table 5. Critical headway without exiting vehicles

Sites	NCHRP Method 1[1]			NCHRP Method 2[2]			NCHRP Method 3[3]		
	t_c (s)	n	std. dev.	t_c (s)	n (% of NCHRP Method 1)	std. dev.	t_c (s)	n (% of NCHRP Method 1)	std. dev.
ALP01-SB	6.018	321	2.2	5.299	56 (17%)	2.3	3.623	7 (2%)	1.4
COV01-SB	6.533	330	1.5	3.464	4 (1%)	1.8	n/a	0 (0%)	n/a
COV01-NB	6.784	263	2.0	4.366	12 (4%)	2.9	4.918	8 (3%)	3.1
COV01-WB	4.957	471	2.0	4.330	129 (27%)	1.9	4.263	10 (2%)	1.1
COV01-EB	4.932	370	2.0	4.949	232 (62%)	2.1	7.028	33 (9%)	2.0
COL01-SEB	6.145	222	2.1	4.382	16 (7%)	2.1	4.318	6 (3%)	1.0
COL01-SWB	5.414	157	2.0	3.781	17 (11%)	2.1	n/a	0 (0%)	n/a
DOU01-EB	6.367	131	2.3	4.876	12 (9%)	2.6	n/a	0 (0%)	n/a
DOU01-WB	5.290	162	1.8	4.980	10 (6%)	2.0	n/a	0 (0%)	n/a
DOU01-SB	6.023	34	2.7	n/a	0 (0%)	n/a	n/a	0 (0%)	n/a
EMO01-SEB	5.555	877	1.7	5.175	284 (32%)	1.7	5.134	157 (18%)	1.5
FAY01-EB	6.108	246	2.2	4.938	33 (13%)	2.4	3.814	7 (3%)	0.9
FAY01-SB	5.664	216	2.0	4.974	25 (12%)	2.3	4.136	3 (1%)	2.0
FAY01-NB	6.327	221	2.1	5.291	21 (10%)	2.4	n/a	0 (0%)	n/a
DUL01-EB	7.852	13	2.3	n/a	0 (0%)	n/a	n/a	0 (0%)	n/a
HIN01-WB	6.204	51	2.5	5.244	13 (25%)	2.5	n/a	0 (0%)	n/a
HIN01-SB	7.111	65	2.1	6.339	3 (5%)	1.2	n/a	0 (0%)	n/a
HOL01-EB	5.183	125	2.2	4.876	39 (31%)	2.0	n/a	0 (0%)	n/a
HOL01-NB	4.621	362	1.9	4.487	176 (49%)	1.8	3.038	4 (1%)	1.4
VIL01-SWB	6.667	184	1.9	4.521	6 (3%)	1.6	5.333	2 (1%)	0.7
NEW01-EB	5.974	117	2.5	4.883	9 (8%)	2.6	n/a	0 (0%)	n/a
NEW01-WB	5.587	221	1.8	4.538	16 (7%)	2.2	n/a	0 (0%)	n/a
ROS01-EB	6.417	167	1.6	3.736	4 (2%)	1.3	n/a	0 (0%)	n/a
ROS01-SWB	4.138	538	1.8	4.230	104 (19%)	1.7	4.530	38 (7%)	1.6
ROS02-EB	6.234	152	1.9	4.557	2 (1%)	2.8	n/a	0 (0%)	n/a
ROS02-SWB	4.443	660	1.7	4.064	112 (16%)	1.6	4.032	58 (9%)	1.5
STS01-WB	6.567	29	3.0	7.733	9 (31%)	2.6	n/a	0 (0%)	n/a
STS01-EB	7.079	19	2.7	n/a	0 (0%)	n/a	n/a	0 (0%)	n/a
Total		6724			1344 (20%)			333 (5%)	
Average (weighted)	5.503		1.916	4.747		1.922	4.922		1.562

Legend: n = number of observations; t_c = critical headway; std. dev. = standard deviation

[1]All observations of gap acceptance (accepted/rejected gaps and rejected lags)

[2]Observations that include a rejected gap

[3]Observations that include a rejected gap and occur during user defined queuing periods > 1 min

The critical headway values used in the capacity equations are calculated according to NCHRP's critical headway Method 2. "Method 2 is the recommended methodology" in NCHRP Report 572 [1]. Method 2 ensures that a driver rejected a gap, thus having to stop and wait to proceed into the intersection, while allowing a higher number of vehicles to be included in the analysis by removing the queuing criteria of Method 3. However, it is seen that there is relative stability between the Method 2 and Method 3 average weighted critical headway values, with a difference of less than 0.2 seconds. For comparative purposes Table 6 shows the Method 2 average, weighted average, and median critical headway values across all sites. The weighted average critical headway value of 4.747 seconds without exiting vehicles was used in the calibrated equations.

Table 6. Critical headway values for all sites using NCHRP Report 572 critical headway Method 2

	All sites, Without exiting
Average (s)	4.445
Weighted average (s)	**4.747**
Median (s)	4.938
N	1344

4.3 FOLLOW-UP HEADWAY

For each roundabout site, follow-up headway values were found using NCHRP Report 572's queued data method and move-up time method. Figure 28 displays the frequency of move-up times without exiting vehicles under queued conditions. The 95th percentile move-up time is 4.0 seconds.

Table 7 provides the observed follow-up headway values for the queued data and move-up time methods. As expected the move-up time method increases the number of follow-up headway observations in this study by approximately 40%.

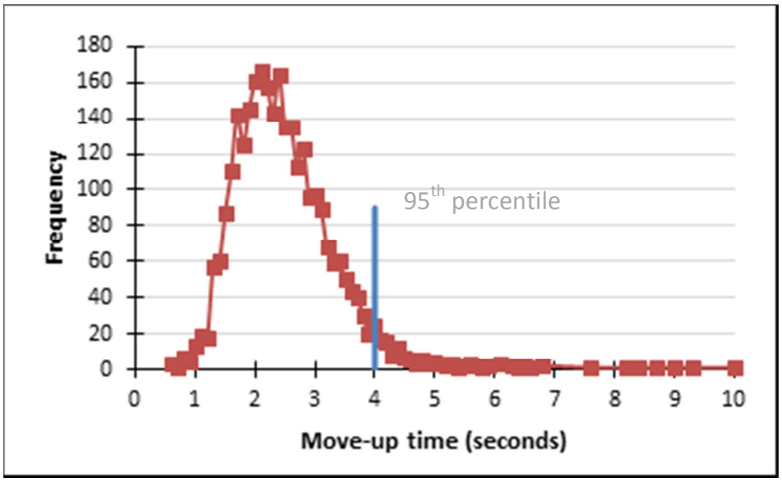

Figure 28. Move-up time frequency of queued data without exiting vehicles (n=2886)

The follow-up headway values that used in the recommended capacity equations were calculated using NCHRP's move-up time method. This method was selected given the insufficient data in the queued data method. Table 8 shows the average, weighted average, and median follow-up headway values for all sites. As with critical gap, the weighted average follow-up headway value is used in the calibrated equations.

Table 7. Follow-up headway excluding exiting vehicles

Site	Queued Data			Move-up Time < 4.0 sec.		
	n	t_f (s)	std. dev.	n	t_f (s)	std. dev.
ALP01-SB	18	4.035	1.8	248	3.412	1.1
COV01-SB	1167	3.477	1.3	1339	3.348	1.1
COV01-NB	637	3.792	1.5	1059	3.687	1.4
COV01-WB	11	3.16	1.6	169	2.966	1.4
COV01-EB	1	1.129	n/a	5	2.655	0.9
COL01-SEB	6	3.701	1.4	158	3.375	1.0
COL01-SWB	22	3.556	1.3	397	3.38	1.1
DOU01-EB	0	n/a	n/a	94	2.81	0.7
DOU01-WB	16	3.481	0.9	258	2.931	0.8
DOU01-SB	0	n/a	n/a	54	3.108	1.1
EMO01-SEB	85	3.897	1.5	133	3.512	0.9
FAY01-EB	24	3.226	1.5	259	3.259	1.3
FAY01-SB	25	3.361	1.8	244	3.078	1.0
FAY01-NB	14	4.027	0.7	248	3.414	1.1
DUL01-EB	0	n/a	n/a	146	3.088	1.0
HIN01-WB	0	n/a	n/a	18	3.635	0.9
HIN01-SB	0	n/a	n/a	120	3.317	1.0
HOL01-EB	0	n/a	n/a	51	3.107	0.6
HOL01-NB	3	3.882	0.7	42	3.137	0.7
VIL01-SWB	133	2.812	1.0	825	2.878	0.9
NEW01-EB	10	3.892	2.5	172	3.167	0.9
NEW01-WB	52	3.242	0.9	368	3.409	1.2
ROS01-EB	102	3.624	1.5	505	3.327	1.1
ROS01-SWB	132	3.19	1.5	474	2.913	1.0
ROS02-EB	92	3.747	1.7	352	3.447	1.2
ROS02-SWB	336	3.249	1.2	351	3.061	1.0
STS01-WB	0	n/a	n/a	30	2.843	0.9
STS01-EB	0	n/a	n/a	37	2.519	1.0
Total	2886			8156		
Weighted Average (s)		3.502	1.4		3.265	1.0

Table 8. Follow-up headway values for all sites using NCHRP Report 572 move-up time method

	Without exiting, All sites
Average (s)	3.171
Weighted average (s)	**3.265**
Median (s)	3.040
n	8156

4.4 EQUATION CALIBRATION

The 2010 HCM single-lane roundabout capacity equations were calibrated based on the weighted average follow-up and critical headway values across all roundabout approaches as determined in sections 4.2 and 4.3. For comparison a calibrated capacity equation for each roundabout approach, based on that approach's follow-up and critical headway, was also determined. Figure 29 shows the calibrated roundabout capacity equations (excluding exiting vehicles) for each of the site locations. The legend is listed in the order of the highest to lowest entry capacity at the conflicting flow of 1500 vph. The dashed line represents the calibrated model using the overall weighted average critical and follow-up headway values of 4.747 and 3.265 seconds respectively.

The calibrated equation is shown below as Equation 15.

$$c_{e,pce} = 1103e^{(-0.0009 \times v_{c,pce})} \quad \textbf{(15)}$$

Where:

$c_{e,pce}$ = capacity of the approach lane under consideration in passenger car equivalents, veh/h

$v_{c,pce}$ = conflicting flow in passenger car equivalents, veh/h

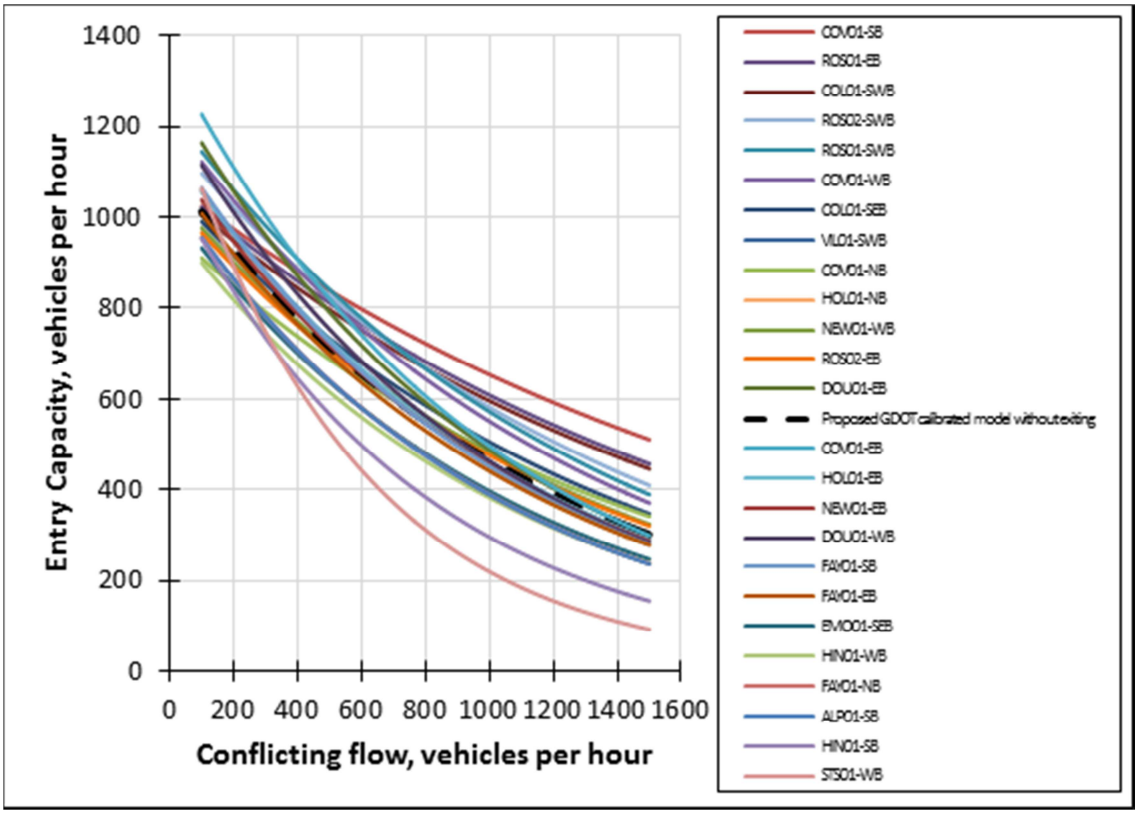

Figure 29. Calibrated single-lane roundabout capacity equations excluding exiting vehicles by approach

4.5 COMPARISON

Figure 30 compares the Georgia-specific calibrated equations excluding exiting vehicles, with exiting vehicles (see Appendix F for calibration details), and the current HCM 2010 capacity model as a function of conflicting vehicle flow. Models including exiting vehicles are considered in the section to allow for a more thorough discussion of the results.

As seen in Figure 30, the proposed model without exiting vehicles is very similar to the HCM 2010 model, predicting a slightly higher capacity than the HCM 2010 model except at very low conflicting volumes. It is also seen that that model including exiting

vehicles appears to predict higher capacities. However, caution must be exercised in directly comparing the proposed model with exiting vehicles to the HCM 2010 capacity model which is based on a calibration excluding exiting vehicles. For a given set of roundabout flows the conflicting flow will differ for each model dependent upon the percentage of conflicting vehicles that are exiting vehicles. For example, as shown in

Table 9, 78% of COV01-SB's conflicting vehicles are exiting vehicles. The capacity prediction for COV01-SB without exiting vehicles is 951 vph. When exiting vehicles are included in the capacity model the capacity prediction decreases to 715 vph. However, for HOL01-NB only 27% of the conflicting volume is exiting vehicles resulting in the capacity prediction increases when including exiting vehicles, from 647 vph to 676 vph.

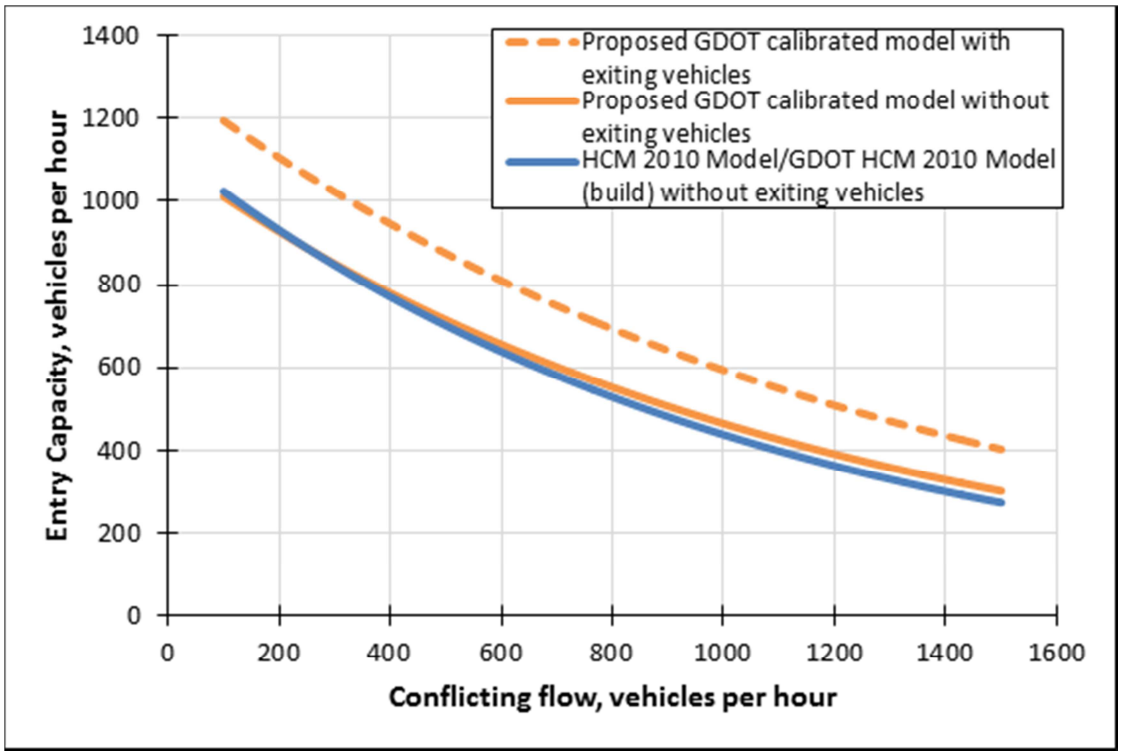

Figure 30. Existing HCM 2010 model and proposed calibrated capacity equations

Table 9. Comparison of capacities for COV01-SB and HOL01-NB

Site		COV01-SB	HOL01-NB
Conflicting Vehicles	Circulating Vehicles (vph)	165	592
	Exiting Vehicles (vph)	573	217
	Total (vph)	738	809
Percent conflicting vehicles that are exiting vehicles (%)		78	27
Capacity without exiting vehicles (vph)		951	647
Capacity with exiting vehicles (vph)		715	676

Table 10 provides the follow-up and critical headway values for the HCM 2010 capacity model, the calibrated models from Bend, Oregon, Caltrans, and Wisconsin, and the proposed calibrated models. Figure 31 displays curves for all of the capacity equation models listed in

Table 10. Except for the Bend, Oregon data, at the higher conflicting flows all capacity models including exiting vehicles are shifted up and to the right relative to the models without exiting vehicles. Recall this does not necessarily imply higher capacities as exiting vehicles increases the total conflicting vehicle volume. Interestingly the Bend, Oregon (without exiting vehicles) and the proposed model with exiting vehicles curves are very similar. There are numerous potential reasons for this, such as an increase in aggressiveness or familiarity with roundabouts of the Oregon drivers (e.g. there are 25 roundabouts in the City of Bend alone whereas there are approximately 100 in the entire state of Georgia [28]); sampling bias in one or both studies (e.g. lower or higher representation of roundabouts with significant exiting vehicles); underlying differences in the design of Georgia and Oregon roundabouts; or other unknown factors. Additional study is necessary to confirm the underlying reason for this difference.

Table 10. Follow-up and critical headway values for current and proposed models

Model	Follow-up Headway (seconds)	Critical Headway (seconds)	Exiting vehicles considered?
Proposed GDOT calibrated model with exiting vehicles	2.788	4.192	Yes
Proposed GDOT calibrated model without exiting vehicles	3.265	4.747	No
Bend, Oregon/ GDOT Calibrated Model (future)	2.7	4.1	No
Caltrans	2.5	4.8	No
HCM 2010 Model/GDOT HCM 2010 Model (build)	3.2	5.0	No
Wisconsin			
Canal Street at 25th St.	2.6	5.5	No
	2.3	4.6	Yes
Sth 78 at CTH ID	3.8	4.8	No
	3.1	3.8	Yes

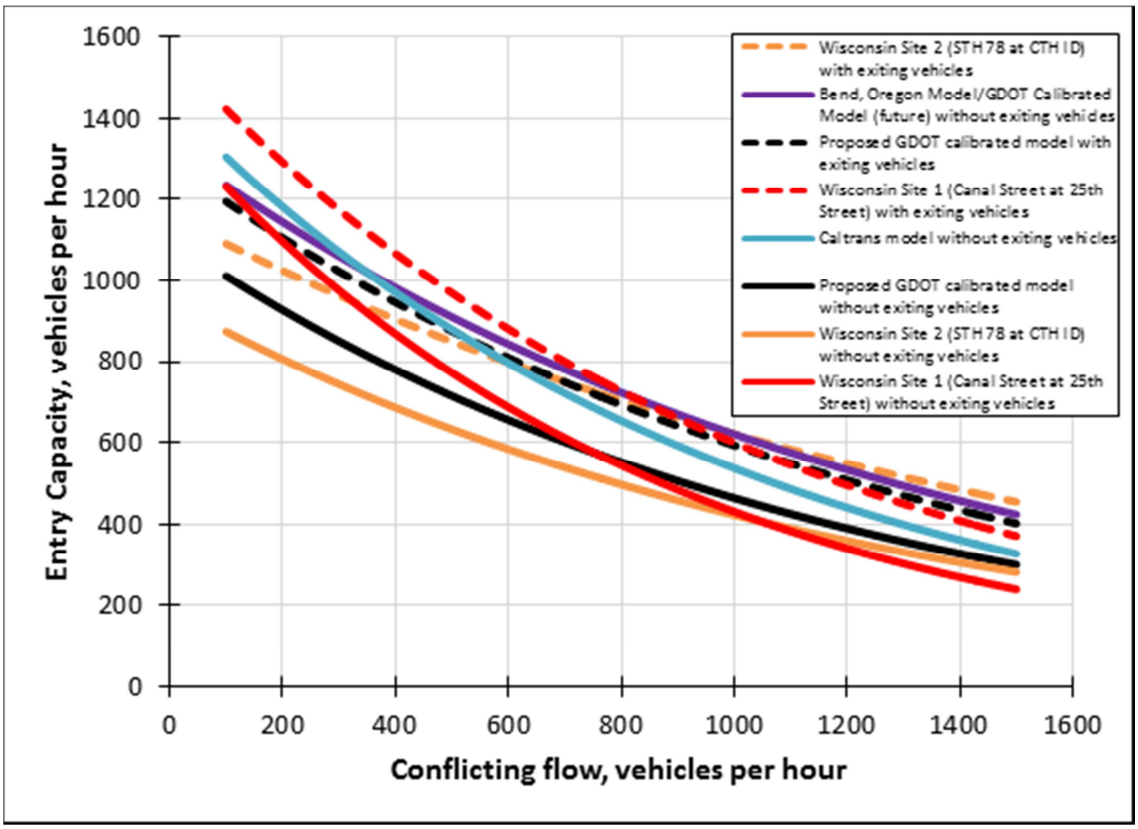

Figure 31. Comparison of capacity equation models

4.6 MODIFIED LIST

As an exploration of the sensitivity of the Georgia calibration to potential non-consistent modern roundabout geometry or unique features a subset of sites was selected with the most consistent design. For example, the EMO0-SEB roundabout site shown in Figure 32 has a large slope which makes it a unique roundabout site. The sites with consistent modern-roundabout features and limited unique features were compiled in a modified list.

Figure 32. Large slope at roundabout in Atlanta, Georgia (Source: Google Earth™, accessed October 22, 2013)

The study examined if the Modified List would yield different headway values than the list including all the sites. Table 11 displays the data collection sites where the shaded rows indicate roundabouts included on the modified list. The rows that are not shaded indicate locations that either do not have consistent modern roundabout geometry or have unique features. The modified list includes the six roundabout sites with data collected on 13 approaches.

Table 11. Roundabout data collection sites

Site	City	Lane Configuration			Collection Date	Approach	Time Duration (hr)	Site ID.
		Circulating Lanes	Legs	Slip Lanes				
Douglas Rd./Southlake Dr./Leeward Walk Cir.	Alpharetta	1	4	0	11/13/2012	SB	1:57	ALP01-SB
Turner Lake Rd. SW/Clark St. SW	Covington	1	4	0	3/1/2012	SB	2:33	COV01-SB
						NB	2:24	COV01-NB
					5/24/2012	WB	2:25	COV01-WB
						EB	2:11	COV01-EB
Warm Springs Rd./Blackmon Rd	Columbus	1	4	0	11/2/2012	SEB	1:39	COL01-SEB
						SWB	2:07	COL01-SWB
SR 166 (Duncan Memorial Hwy)/SR 5 (Bill Arp Rd.)	Douglasville	1	4	2	5/14/2012	EB	2:17	DOU01-EB
						WB	2:10	DOU01-WB
					11/1/2012	SB	1:27	DOU01-SB
N. Decatur Rd./Oxford Rd NE.	Atlanta	1	5	1	10/19/2012	SEB	1:49	EMO01-SEB
Grady Ave./Beauregard Blvd.	Fayetteville	1	4	1	4/11/2012	EB	2:04	FAY01-EB
						SB	2:02	FAY01-SB
					10/23/2012	NB	2:19	FAY01-NB
McClure Bridge Rd./W. Lawrenceville St./Irvindale Rd. NW	Duluth	1	3	0	6/1/2012	EB	1:49	DUL01-EB
N. Main St./Memorial Dr.	Hinesville	1	4	0	7/27/2012	WB	1:11	HIN01-WB
						SB	1:39	HIN01-SB
Holly Springs Rd./Davis Rd.	Marietta	1	4	0	10/11/2012	EB	1:58	HOL01-EB
						NB	1:52	HOL01-NB
Villa Rica/Sandtown	Marietta	1	4	0	3/27/2012	SWB	1:52	VIL01-SWB
E. Broad St./Greison Tr./E. Newnan Rd.	Newnan	1	4	0	10/25/2012	EB	1:55	NEW01-EB
						WB	1:52	NEW01-WB
Grimes Bridge Rd./Norcross St./Warsaw Rd./Melody Ln.	Roswell	1	5	2	5/15/2012	EB	2:06	ROS01-EB
						SWB	2:06	ROS01-SWB
					10/26/2012	EB	2:00	ROS02-EB
						SWB	2:00	ROS02-SWB
Lawrence Rd./Frederica Rd.	St. Simons	1	3	0	7/28/2012	WB	2:52	STS01-WB
						EB	2:08	STS01-EB
						Total	**56:44:00**	

Modern roundabout geometry

Figure 33 displays the calibrated roundabout capacity equations excluding exiting vehicles for each of the site locations on the modified list. The legend for this figure is listed in the order of the highest to lowest entry capacity at the conflicting flow of 1500 vph. The dashed line represents the proposed calibrated model using the overall weighted average critical and follow-up headway values of 4.738 and 3.312 seconds respectively.

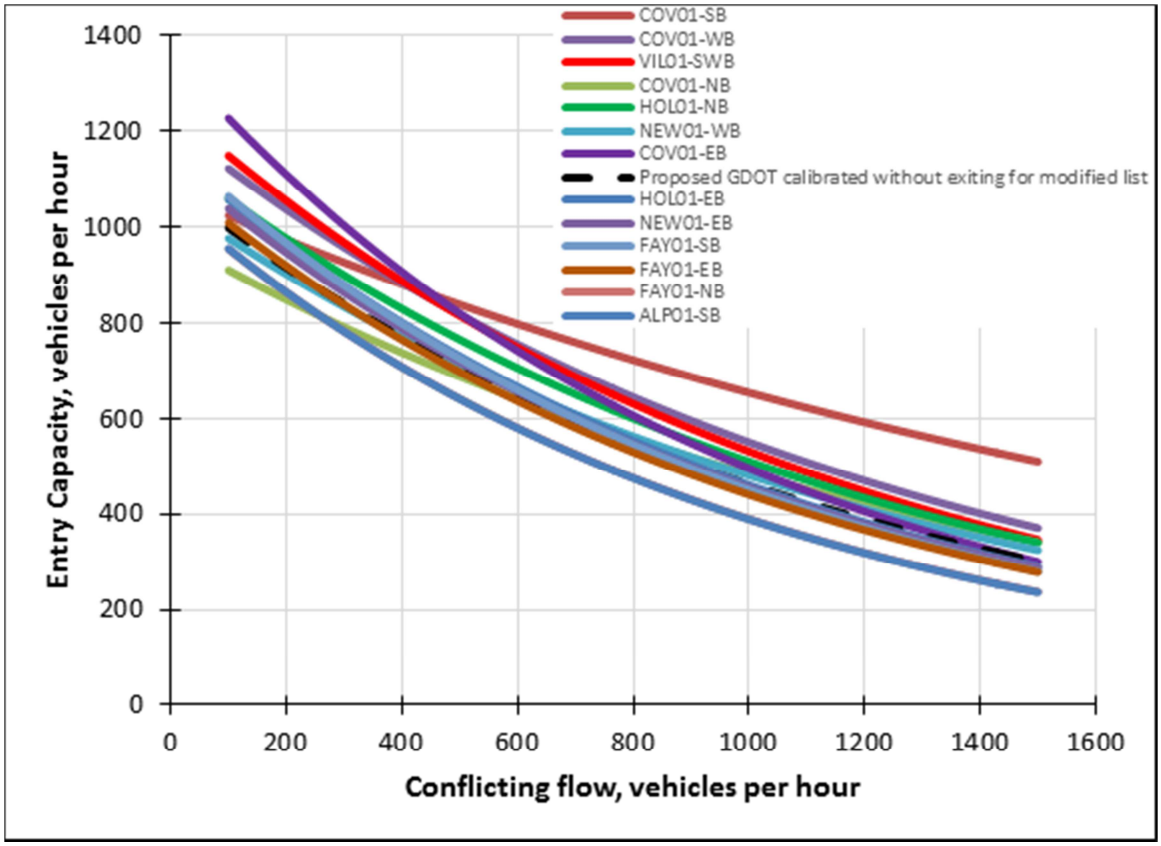

Figure 33. Calibrated single-lane roundabout capacity equations excluding exiting vehicles for modified list

The calibrated equation for the modified List excluding exiting vehicles is shown below as Equation 18.

$$c_{e,pce} = 1087e^{(-0.0009 \times v_{c,pce})} \quad \textbf{(16)}$$

Where:

$c_{e,pce}$ = capacity of the approach lane under consideration in passenger car equivalents, veh/h

$v_{c,pce}$ = conflicting flow in passenger car equivalents, veh/h

Table 12 shows there is an insignificant difference between the critical headway values from the list with all site locations and the modified list. Likewise, Table 13 shows there is an insignificant difference between the follow-up headway values from the list with all site locations and the modified list, with a less than a 0.1 second difference between the headway values. Figure 33 further shows the limited difference between the modified list and full set of sites in Figure 29. These results suggest that difference in geometry between the modified list roundabouts and the other roundabouts was not sufficient to affect the operations of the roundabout. Thus, it is recommended to use the calibrated equations based on all 28 roundabout approaches.

Table 12. Critical headway values for modified site list using NCHRP Report 572 critical headway Method 2

	All sites, Without exiting	Modified list, Without exiting
Average (s)	4.445	4.686
Weighted average (s)	4.747	4.738
Median (s)	4.938	4.876
Number of observations	1344	758

Table 13. Follow-up headway values for modified site list using NCHRP Report 572 move-up time method

	All Sites, Without exiting	Modified List, Without exiting
Average (s)	3.171	3.194
Weighted average (s)	3.265	3.312
Median (s)	3.040	3.084
Number of observations	8156	5029

CHAPTER 5: GDOT ROUNDABOUT ANALYSIS TOOL

The GDOT Roundabout Analysis Tool was updated to reflect the results of the Georgia-specific (local) calibration discussed above. The previously discussed (without exiting vehicles) roundabout capacity parameters were incorporated into the existing GDOT Analysis Tool to create a draft version of locally calibrated roundabout capacity model. The draft GDOT Analysis Tool Excel workbook is labeled "Draft without exiting.xlsx". In addition, a second Microsoft Excel® workbook has been created "Draft with exiting.xlsx" that reflects the Appendix F roundabout model calibration including exiting vehicles. For the "Single Lane" worksheet in the "Draft without exiting.xlsx" workbook the "HCM 2010 Model (Build)" Entry Capacity equation has been edited to reflect the Georgia calibrated roundabout capacity parameters in Equation 15. For the "Single Lane" worksheet in the "Draft with exiting.xlsx" workbook the "Entry/Conflicting Flows" Conflicting Flow formulas and the "HCM Model 2010 (build)" entry capacity have been updated to reflect the "Georgia-calibrated with exiting vehicles" model described in Appendix F. Parameters from Appendix F, Equation 17 were used to update the "HCM 2010 Model (Build)" Entry Capacity equation in the existing model. The conflicting flow formulas were updated to account for exiting vehicles, with the conflicting flow the sum of circulating vehicles and exiting vehicles at the approach of interest.

In addition, in both Microsoft Excel® files the "Calibrated Model (future)" analysis sections in the "Single Lane" worksheet were updated to allow for a sensitivity analysis based on the study results. The "Calibrated Model (future)" analysis was given an additional option labelled "Future Percentile" that allows the user to select capacity

values based on the 15th, 55th, 65th, 75th, and 85th percentile capacities of the Georgia data rather than a fixed (default) value. For the "Draft with exiting.xlsx" workbook the conflicting flows are also updated as discussed above.

The percentile capacity values were found through a Monte Carlo analysis using the study data. To conduct this analysis, a set of 10,000 pairs of critical and follow-up headway values were generated based on the Georgia data. For each critical and follow-up headway pair, inferred capacities were determined for all conflicting flows ranging from 0 to 1600 vph in steps of 50 vph. The resulting 10,000 capacity estimates were rank ordered and the 15th, 55th, 65th, 75th, and 85th capacities were determined for each value of the conflicting flow. Finally, the capacity equation parameter set representative of each of the percentile capacities was determined.

Designers may use these percentile values where, based on engineering judgment, they believe that capacities would be higher (or lower) in a given area either now or in the future than the capacity based on the current "HCM2010 Model (build)" Georgia calibration. These values allow for a sensitivity analysis of a design given increased capacities due to potential increased driver aggressiveness or familiarity with roundabout operations.

Finally, a new worksheet has been added to both workbooks labeled "Capacity Sensitivity Table." This worksheet contains a table with the calculated Georgia calibrated single lane capacity equation capacities for percentile values of 15th, 55th, 65th, 75th, and 85th as well as the "HCM 2010 Model (build)" Georgia Calibration. Capacities are found for each percentile at conflicting flows ranging from 0 pcph to 1600

pcph in 50 pcph increments. This table is intended as informational to the designer, allowing for a quick assessment of the sensitivity of the Single Lane Capacity equation to increasing conflicting flows and higher parameter values.

CHAPTER 6: CONCLUSIONS

One aspect of determining if a roundabout is a feasible intersection treatment is to perform an operational analysis through the use of one or more intersection capacity models. To assist in these analyses, the GDOT Roundabout Analysis Tool incorporates two different single-lane roundabout capacity models. Currently the first of these models is identical to the default single-lane roundabout capacity equation found in the 2010 Highway Capacity Manual (HCM) while the second model uses the same capacity equations with different parameter values calibrated with follow-up and critical headway values derived from studies in California and Bend, Oregon. The purpose of this study was to measure follow-up and critical headways at Georgia roundabouts in order to calibrate the 2010 HCM capacity equations to yield improved capacity predictions.

This study closely followed the methodology of the NCHRP Report 572. The NCHRP Report 572 presents several methods for calculating both follow-up and critical headway. The research team filmed 28 approaches at thirteen Georgia roundabouts for a total of 56.5 hours of data. Time stamp data was extracted at key points as described in NCHRP Report 572. Follow-up headways were calculated using both the "queued data" and "move-up time" methods. Critical headways were calculated using Methods 1, 2, and 3 presented in NCHRP Report 572. Consistent with this earlier report, the final calibrated models incorporated the critical headway determined by Method 2 and the follow-up headway determined by the "move-up time" method. Lastly, this study analyzed the impact of including exiting vehicles in the roundabout analysis by calculating follow-up and critical headway with exiting vehicle data.

The critical and follow-up headways determined by analysis of the study data (excluding exiting vehicles) were found to be 4.747 seconds and 3.265 seconds,

respectively. These are similar to the HCM 2010 default values for critical and follow-up headway of 5.0 and 3.2 seconds, respectively. This study's calibrated model excluding exiting vehicles predicts slightly higher capacity than the 2010 HCM model except at conflicting volumes of 300 vehicles per hour or less.

Also, as seen in Appendix F, this study found that vehicles exiting the roundabout immediately before the conflict zone were likely to impact capacity and were not considered in the existing analysis. The study found that the percentage of conflicting vehicles that were exiting vehicles influenced the change in the headway values. At sites with a large percentage of exiting vehicles, the follow-up headway values had the largest decrease when exiting vehicles were included in the analysis. Therefore, this study suggests, in contrast to the previous NCHRP studies, that both the presence and proportion of exiting vehicles impacts the capacity and may need to be considered under certain conditions.

Lastly, this study determined the follow-up and critical headway values for sites with consistent modern roundabout features and limited unique features for comparison with other designs. Six of the thirteen roundabouts studied fit into this category. The critical and follow-up headway values for these roundabouts excluding exiting vehicles were 4.738 and 3.312 seconds respectively, a less than a 0.1 seconds difference between the headway values when using all sites. This result suggests that the geometric differences between the subset of modern roundabouts and the full selection does not significantly affect the operational capacity of the roundabout. However, this finding should not be interpreted as modern roundabout geometry is not effective, only that the roundabouts in this study were not sufficiently different to impact capacity. Also, this analysis does not consider the potential safety benefits of the modern roundabout geometry as this was outside the scope of this project.

6.1 LIMITATIONS

One limitation of the study is the age of the included roundabouts. A majority of the roundabouts studied were built in the last five years and thus drivers may still be adjusting to navigating roundabouts. Also, with the exception of the Covington and Roswell roundabouts no other sites had consistent queuing. Therefore, only data from Covington and Roswell was captured under consistently capacity constrained conditions. Thus, this study recommends a follow-on calibration effort be performed in approximately 5 years to determine if there was lack-of-driver-familiarity bias reducing observed capacities and potentially observe additional roundabout approaches under queued conditions given expected growth in traffic volumes.

A second limitation is that the current critical and follow-up headway analysis does not distinguish between passenger cars and heavy vehicles in the video data processing. It is expected that this will have minimal impact on the calibrated equations given: the low percentage of heavy vehicles, the tendency of the move-up time method to eliminate any truck following truck data, and the minimal impact of the truck data on the maximum likelihood method. However, future efforts should investigate the potential impact on the calibration and address any bias if necessary.

Finally, as noted in the QA/QC section of this report the research team for NCHRP Report 572 collected all keystrokes timestamps in one pass through the video. This study chose not follow the NCHRP method as preliminary testing found it very difficult to accurately capture all keystrokes in real time in one pass through the video. Rather, this study elected to collect the keystrokes over three passes through the same video. However, as mentioned, while allowing for increased accuracy in data collection new challenges were created in the merging of the processed data. Future efforts

should consider alternative data collection methods, such as all keystrokes are collected in one pass through a video shown at less than real-time speed or the use of automation of data collection through software.

6.2 RECOMMENDATIONS

1) The primary objective of this effort was the calibration of the roundabout capacity equation. Thus, following NCHRP Report 572 methods, the single lane capacity equation is recommended as:

$$c_{e,pce} = 1103e^{(-0.0009 \times v_{c,pce})} \quad \textbf{(15)}$$

$c_{e,pce}$ = capacity of the approach lane under consideration in passenger car equivalents, veh/h

$v_{c,pce}$ = conflicting flow in passenger car equivalents, veh/h

A draft Georgia Roundabout Analysis Toolbox that has been updated to reflect this calibrated equation has been provided separately.

2) For estimation of future conditions it is recommended that the percentiles capacities based on Georgia data be utilized rather than the Bend, Oregon based equation. These capacity percentiles provide an estimated capacity based on the higher performing approaches in Georgia and are likely more representative of future expected Georgia conditions. Capacities are provided for 55th, 65th, 75th, and 85th percentiles. Engineering judgment is required in the selection of which percentile best represents potential future conditions for each roundabout location. However, it is expected that higher percentiles would be used in areas where higher concentrations of roundabouts and higher volume demands are expected. Both factors leading to increased driver familiarity with, and aggressiveness in, roundabouts.

3) Findings from the analysis including exiting vehicles in the calibration strongly suggest the potential for exiting vehicles as a critical factor on at least some

roundabouts. It is likely that the proportion of exiting vehicles in the circulating traffic, along with geometric features or other potential variables influences the approach capacity. It is recommended that potential alternative models be explored allowing for the inclusion of exiting vehicles in the capacity analysis.

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APPENDIX A: FOLLOW-UP HEADWAY EXAMPLES

A.1 FOLLOW-UP HEADWAY EXAMPLE

"The follow-up headway, t_f, is defined as the headway maintained by two consecutive entering vehicles using the same gap in the conflicting stream" – NCHRP Report 572.

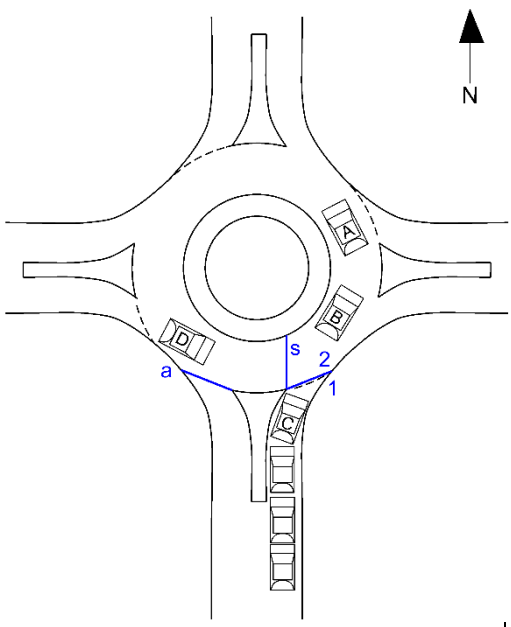


Figure 34. Schematic for follow-up headway example

ASSUMPTIONS:

1. Constant queuing is present on the south leg of the roundabout
2. Vehicle A is circulating in the roundabout and crosses line "s" at t = 0 sec.
3. Vehicle B enters the roundabout and crosses line "2" at t = 2 sec.
4. Vehicle C enters the roundabout and crosses line "2" at t = 4 sec.
5. Vehicle D is circulating in the roundabout and crosses line "s" at t = 7 sec.

Sample calculation for the follow-up headway between Vehicles B and C:

$$t_f = C_2 - B_2$$
$$t_f = 4\ sec. - 2\ sec.$$
$$t_f = 2\ sec.$$

Where:

$t_f = follow - up\ headway, sec.$

$C_2 = timestamp\ when\ Vehicle\ C\ crosses\ line$ "2"

$B_2 = timestamp\ when\ Vehicle\ B\ crosses\ line$ "2"

A.2 CRITICAL HEADWAY NCHRP METHOD 1 EXAMPLE

Method 1 is the inclusion of all observations of gap acceptance, including rejected lags.

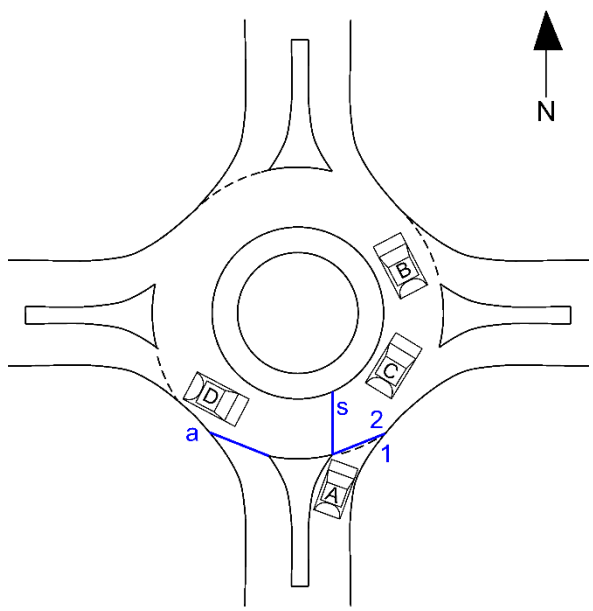


Figure 35. Schematic for critical headway NCHRP Method 1 example

ASSUMPTIONS:

1. Vehicle A arrives at the roundabout and stops at line "1" at t = 0 sec.
2. Vehicle B is circulating the roundabout and crosses line "s" at t = 1 sec.
3. Vehicle C is circulating in the roundabout and crosses line "s" at t = 3 sec.
4. Vehicle A enters the roundabout and crosses line "2" at t = 6 sec.
5. Vehicle D is circulating in the roundabout and crosses line "s" at t = 8 sec.

Sample calculations for the accepted and rejected gaps and lags:

1. Rejected lag between Vehicle A and Vehicle B

$lag = B_s - A_1$
$lag = 1\ sec. - 0\ sec.$
$lag = 1\ seconds$

Where:
$B_S = timestamp\ when\ Vehicle\ B\ crosses\ line\ "s"$
$A_1 = timestamp\ when\ Vehicle\ A\ arrives\ at\ line\ "1"$

2. Rejected gap between Vehicle B and Vehicle C

$gap = C_s - B_s$
$gap = 3\ sec. - 1\ sec.$
$gap = 2\ seconds$

Where:
$C_S = timestamp\ when\ Vehicle\ C\ crosses\ line\ "s"$
$B_S = timestamp\ when\ Vehicle\ B\ crosses\ line\ "s"$

3. Accepted gap between Vehicle C and Vehicle D

$gap = D_s - C_s$
$gap = 8\ sec. - 3\ sec.$
$gap = 5\ seconds$

Where:
$D_S = timestamp\ when\ Vehicle\ D\ crosses\ line\ "s"$
$C_S = timestamp\ when\ Vehicle\ C\ crosses\ line\ "s"$

A.3 CRITICAL HEADWAY NCHRP METHOD 2 EXAMPLE

Method 2 is the "inclusion of only observations that contain a rejected gap" – NCHRP Report 572.

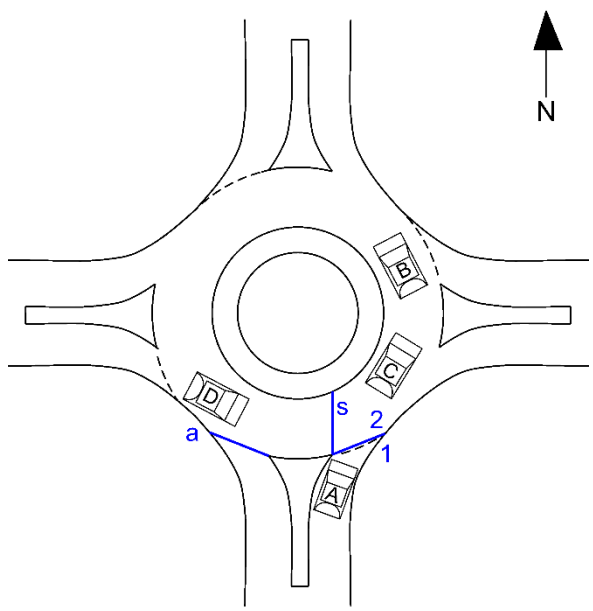


Figure 36. Schematic for critical headway NCHRP Method 2 example

ASSUMPTIONS:

1. Vehicle A arrives at the roundabout and stops at line "1" at t = 0 sec.
2. Vehicle B is circulating the roundabout and crosses line "s" at t = 1 sec.
3. Vehicle C is circulating in the roundabout and crosses line "s" at t = 3 sec.
4. Vehicle A enters the roundabout and crosses line "2" at t = 6 sec.
5. Vehicle D is circulating in the roundabout and crosses line "s" at t = 8 sec.

Sample calculations for the accepted and rejected gaps:

1. Rejected gap between Vehicle B and Vehicle C

$gap = C_s - B_s$
$gap = 3\ sec. - 1\ sec.$
$gap = 2\ seconds$

Where:
$C_S = timestamp\ when\ Vehicle\ C\ crosses\ line\ "s"$
$B_S = timestamp\ when\ Vehicle\ B\ crosses\ line\ "s"$

2. Accepted gap between Vehicle C and Vehicle D

$gap = D_s - C_s$
$gap = 8\ sec. - 3\ sec.$
$gap = 5\ seconds$

Where:
$D_S = timestamp\ when\ Vehicle\ D\ crosses\ line\ "s"$
$C_S = timestamp\ when\ Vehicle\ C\ crosses\ line\ "s"$

A.4 CRITICAL HEADWAY NCHRP METHOD 3 EXAMPLE

Method 3 is the "inclusion of only observations where queuing was observed during the entire minute and the driver rejected a gap." – NCHRP Report 572

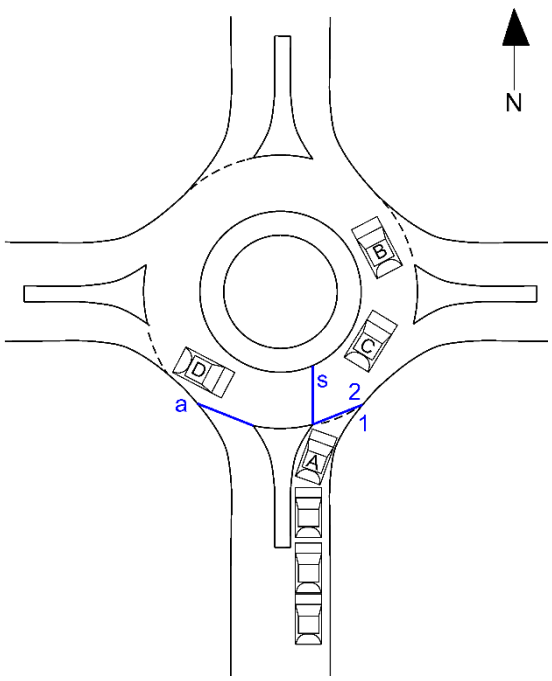


Figure 37. Schematic for critical headway NCHRP Method 3 example

ASSUMPTIONS:

1. Constant queuing is present on the south leg of the roundabout
2. Vehicle A arrives at the roundabout and stops at line "1" at t = 0 sec.
3. Vehicle B is circulating the roundabout and crosses line "s" at t = 1 sec.
4. Vehicle C is circulating in the roundabout and crosses line "s" at t = 3 sec.
5. Vehicle A enters the roundabout and crosses line "2" at t = 6 sec.
6. Vehicle D is circulating in the roundabout and crosses line "s" at t = 8 sec.

Sample calculations for the accepted and rejected gaps:

1. Rejected gap between Vehicle B and Vehicle C

$$gap = C_s - B_s$$
$$gap = 3\ sec. - 1\ sec.$$
$$gap = 2\ seconds$$

Where:
$C_S = timestamp\ when\ Vehicle\ C\ crosses\ line\ "s"$
$B_S = timestamp\ when\ Vehicle\ B\ crosses\ line\ "s"$

2. Accepted gap between Vehicle C and Vehicle D

$$gap = D_s - C_s$$
$$gap = 8\ sec. - 3\ sec.$$
$$gap = 5\ seconds$$

Where:
$D_S = timestamp\ when\ Vehicle\ D\ crosses\ line\ "s"$
$C_S = timestamp\ when\ Vehicle\ C\ crosses\ line\ "s"$

APPENDIX B: PROJECTED TRAVEL TIME EXAMPLE

This example uses Wisconsin's projected travel time method to account for exiting vehicles in the gap/lag measurement.

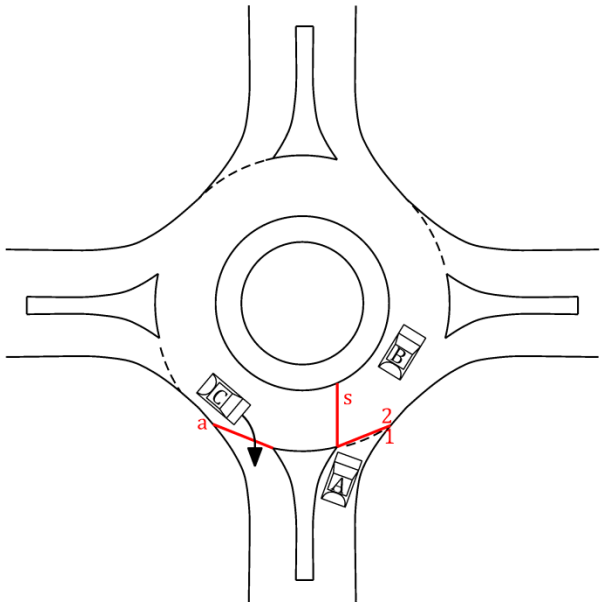


Figure 38. Schematic for projected travel time example

ASSUMPTIONS:

1. Projected travel time, Δt = 2 sec.
2. Vehicle A arrives at the roundabout and stops at line "1" at t = 0 sec.
3. Vehicle B is circulating the roundabout and crosses line "s" at t = 1 sec.
4. Vehicle C is exiting the roundabout and crosses line "a" at t = 5 sec.
5. Vehicle A enters the roundabout and crosses line "2" at t = 6 sec.

Sample calculations for the accepted and rejected gaps/lags:

1. Rejected lag between Vehicle A and Vehicle B

$$lag = T_2 - T_1 + \Delta t$$
$$lag = 1\ sec. - 0\ sec. + 0\ sec.$$
$$lag = 1\ seconds$$

Where:

$T_1 = timestamp\ when\ Vehicle\ A\ arrives\ at\ line\ "1"$

$T_2 = timestamp\ when\ Vehicle\ B\ crosses\ line\ "s"$

2. Rejected gap between Vehicle B and Vehicle C

$$gap = T_2 - T_1 + \Delta t$$
$$gap = 5\ sec. - 1\ sec. + 2\ sec.$$
$$gap = 6\ seconds$$

Where:

$T_1 = timestamp\ when\ Vehicle\ B\ crosses\ line\ "s"$

$T_2 = timestamp\ when\ Vehicle\ C\ crosses\ line\ "a"$

APPENDIX C: ROUNDABOUT DATA COLLECTION INSTRUCTIONS

C.1 OVERVIEW

1. Click on the “JavaProgram_Barry.bat” file.

Located here: Y:\common\GDOT_Roundabouts\Roundabout_Program

2. The program will open an explorer window, navigate to the video for which timestamps will be collected and open the video.
3. Enter your name in the input pop-up window and press ok.
4. The video will open up in the program and two csv files will be created in the same location as the video. The names of the csv files will be videoname.avi_param and videoname.avi_data. The program will write the timestamps into the .avi_data file as they are collected.
5. When the video opens up in the program, it should be at the beginning. There should be lines on the video corresponding to three distinct events. If there are no lines on the video, then it is the wrong video.
6. Each video will need to be watched **three times.**
7. Press play and begin collecting timestamps.
8. The cursor must be flashing in the first box on the bottom of the program window. If the cursor is not in that box, then the program will not collect timestamps.

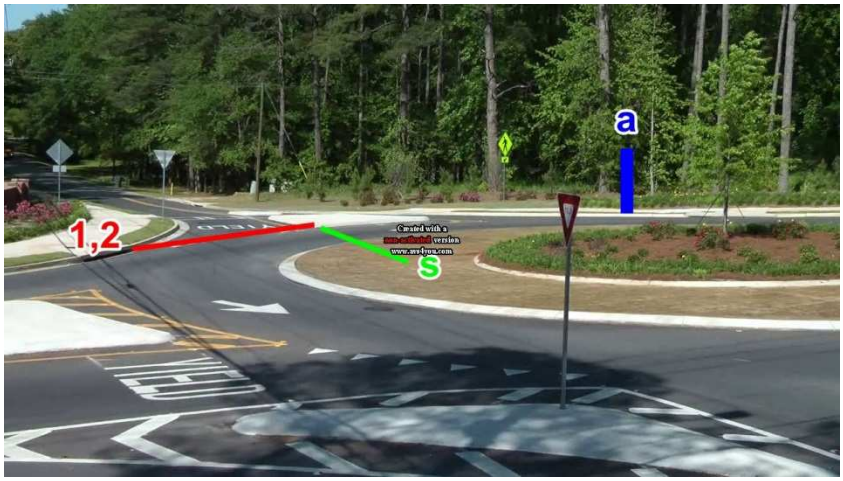


Figure 39. Interface of program

C.2 KEYSTROKES 1 &2

9. For the first review of the video, timestamps corresponding to the arriving and entering event should be collected.
10. The event corresponding to the arrival and entry of a vehicle on the approach of interest is denoted by the red line. The numbers 1 and 2 are shown to the left of this line to remind the collector which keys are to be pressed.
11. Check and make sure that the num lock is on. Timestamps 1 and 2 must be collected using the number pad.
12. “1” is the arrival timestamp. Press “1” when a vehicle arrives on the approach. If the car does not stop then “1” should be pressed when the front of the vehicle reaches the red line. If the vehicle stops then “1” should be pressed when the vehicle stops even if it stops before the red line. Similarly, if the arriving vehicle slows significantly due to a conflict with a circulating vehicle, “1” should be pressed when the vehicle slows its speed significantly.

13. “2” is the departing timestamp. “2” should always be pressed when the vehicle enters the roundabout. When the front of the vehicle crosses the red line “2” should be pressed regardless of where the “1” (arrival) timestamp was collected.
14. When the video ends, close the program. Navigate to the csv files and add “12_” to the beginning of the file name for both of the csv files.
15. Move the csv files into the folder named “Excel Files” that is in the same location as the video file.

C.3 KEYSTROKES A & S

16. For the second review of the video timestamps “a” and “s” should be collected.
17. “a” corresponds to the timestamp for exiting vehicles. The “a” key should be pressed when the front of an exiting vehicle reaches the vertical blue line on the screen.
18. “s” corresponds to the circulating vehicle timestamp. The “s” key should be pressed when the front of a circulating vehicle reaches the green line.
19. When the video ends, close the program. Navigate to the csv files and add “as_” to the beginning of the file name for both of the csv files.
20. Move the csv files into the folder named “Excel Files” that is in the same location as the video file.

C.4 KEYSTROKES X & Z

21. Keystrokes “x” and “z” correspond to queuing data on the approach.
22. There must be at least two vehicles on the approach for a queue to exist.
23. A queue is defined to exist when a vehicle’s speed is determined by the vehicle in front of it. In other words if a vehicle is experiencing delay at the roundabout due

to the vehicle(s) in front of it on the approach, then these vehicles are considered to be queued.

24. “x” should be pressed when queuing begins on the approach. “x” should be pressed at the beginning of the queued conditions even if the queue is only two vehicles long.
25. “z” should be pressed right after the last vehicle in the queue departs the approach.
26. When the video ends, close the program. Navigate to the csv files and add “xz_” to the beginning of the file name for both of the csv files.
27. Move the csv files into the folder named “Excel Files” that is in the same location as the video file.

C.5 DATA COLLECTION ERRORS

1. If an error is made in the data collection, enter keystroke “q”. Continue data collection until you are finished. After you have renamed the files, open the excel file. Delete the rows with mistake keystroke and the keystroke indicator.

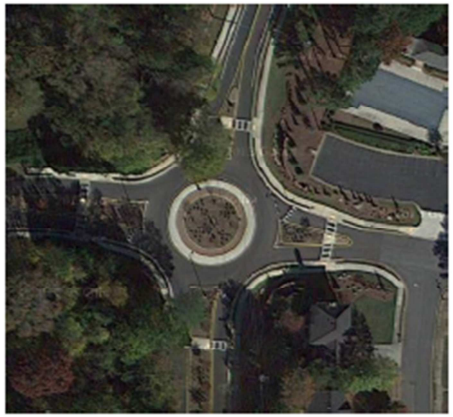
APPENDIX D: GDOT DISTRICT MAP WITH DATA COLLECTION SITES

Figure 40. GDOT District map with data collection sites

APPENDIX E: ROUNDABOUT APPROACH DATA SHEETS

Table 14. Data summary sheet for Alpharetta southbound approach

Site	Alpharetta		
ID	ALP01-SB		
Approach	Southbound		
Intersection	Douglas Rd./Southlake Dr./Leeward Walk Cir.		
County	Fulton		
City	Alpharetta		
GDOT District	7		
AADT	9930		
Date of data collection	Tuesday, November 13, 2012		
Time of data collection	7:18 AM – 9:15 AM		
Video duration	1:57:00		
Queuing periods at least 1 minute long	3		
Total number of queued minutes	3		
	Total data	vph data	
Number of entering vehicles	627	322	
Number of circulating vehicles	640	329	
Number of exiting vehicles	1451	745	



Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	4.405 (2.5)	211	3.930 (3.2)	182	2.464 (0.8)	350	1.841 (1.0)	318
Excluding exiting vehicles	8.966 (5.5)	52	8.459 (7.3)	131	3.006 (1.5)	103	2.043 (1.6)	166

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	2.624 (n/a)	1	3.6	2.813 (0.7)	118
Excluding exiting vehicles	4.035 (1.8)	18	4.0	3.412 (1.1)	248

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	3.734 (1.8)	879	3.467 (1.7)	341	2.960 (1.0)	26
Excluding exiting vehicles	6.018 (3.6)	321	5.299 (5.4)	56	3.623 (1.4)	7

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation
[1]Follow up headway observations during all user defined queuing periods > 1 minute
[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts
[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)
[4]Observations that include a rejected gap
[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 15. Data summary sheet for Covington southbound approach

Site	Covington		
ID	COV01-SB		
Approach	Southbound		
Intersection	Turner Lake Rd. NW/Clark St. SW		
County	Newton		
City	Covington		
GDOT District	2		
AADT	8110		
Date of data collection	Thursday, March 1, 2012		
Time of data collection	4:20 PM – 7:00 PM		
Video duration	2:40:00		
Queuing periods at least 1 minute long		16	
Total number of queued minutes		87	
		Total data	vph data
Number of entering vehicles		1876	704
Number of circulating vehicles		438	165
Number of exiting vehicles		1528	573

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	4.362 (0.9)	280	3.027 (1.2)	579	2.877 (0.9)	294	2.142 (0.9)	634
Excluding exiting vehicles	5.319 (1.1)	11	3.600 (4.0)	100	2.743 (1.0)	73	1.805 (1.4)	246

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	2.778 (0.8)	514	3.6	2.635 (0.6)	415
Excluding exiting vehicles	3.477 (1.3)	1167	4.0	3.348 (1.1)	1339

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	3.943 (1.8)	1208	3.755 (1.1)	213	3.715 (1.1)	186
Excluding exiting vehicles	6.533 (1.5)	330	3.464 (1.8)	4	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 16. Data summary sheet for Covington northbound approach

Site	Covington		
ID	COV01-NB		
Approach	Northbound		
Intersection	Turner Lake Rd. NW/Clark St. SW		
County	Newton		
City	Covington		
GDOT District	2		
AADT	8110		
Date of data collection	Thursday, March 1, 2012		
Time of data collection	4:20 PM – 7:00 PM		
Video duration	2:40:00		
Queuing periods at least 1 minute long	22		
Total number of queued minutes	49		
	Total data	vph data	
Number of entering vehicles	1527	573	
Number of circulating vehicles	390	147	
Number of exiting vehicles	2025	760	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	4.519 (1.2)	367	3.016 (1.6)	510	3.277 (1.2)	360	2.498 (0.9)	679
Excluding exiting vehicles	8.125 (2.8)	21	4.530 (3.2)	97	2.771 (1.4)	46	2.255 (1.6)	196

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	2.736 (0.7)	170	3.6	2.775 (0.7)	241
Excluding exiting vehicles	3.792 (1.5)	637	4.0	3.687 (1.4)	1059

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.109 (1.4)	1406	4.138 (1.6)	327	4.158 (1.4)	171
Excluding exiting vehicles	6.784 (2.0)	263	4.366 (2.9)	12	4.918 (3.1)	8

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 17. Data summary sheet for Covington westbound approach

Site	Covington		
ID	COV01-WB		
Approach	Westbound		
Intersection	Turner Lake Rd. NW/Clark St. SW		
County	Newton		
City	Covington		
GDOT District	2		
AADT	8110		
Date of data collection	Thursday, May 24, 2012		
Time of data collection	4:30 PM – 6:45 PM		
Video duration	2:15:00		
Queuing periods at least 1 minute long	2		
Total number of queued minutes	2		
	Total data	vph data	
Number of entering vehicles	569	253	
Number of circulating vehicles	1282	570	
Number of exiting vehicles	487	217	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	6.518 (3.3)	156	5.335 (4.2)	174	3.092 (1.1)	249	2.058 (1.3)	237
Excluding exiting vehicles	7.970 (5.0)	96	6.161 (5.6)	170	2.916 (1.1)	185	1.784 (1.2)	190

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	2.503 (1.2)	7	3.6	2.575 (0.7)	117
Excluding exiting vehicles	3.160 (1.6)	11	4.0	2.966 (1.4)	169

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.800 (1.9)	642	4.774 (1.7)	222	4.133 (1.3)	11
Excluding exiting vehicles	4.957 (2.0)	471	4.330 (1.9)	129	4.263 (1.1)	10

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 18. Data summary sheet for Covington eastbound approach

Site	Covington		
ID	COV01-EB		
Approach	Eastbound		
Intersection	Turner Lake Rd. NW/Clark St. SW		
County	Newton		
City	Covington		
GDOT District	2		
AADT	8110		
Date of data collection	Thursday, May 24, 2012		
Time of data collection	4:30 PM – 6:45 PM		
Video duration	2:15:00		
Queuing periods at least 1 minute long	1		
Total number of queued minutes	1		
	Total data	vph data	
Number of entering vehicles	305	136	
Number of circulating vehicles	2107	937	
Number of exiting vehicles	722	321	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	5.302 (3.1)	66	4.507 (2.9)	20	2.023 (1.5)	256	1.866 (1.1)	77
Excluding exiting vehicles	6.216 (4.0)	62	4.501 (2.9)	24	1.941 (1.6)	235	1.966 (1.3)	73

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	1.129 (n/a)	1	3.6	2.204 (0.9)	3
Excluding exiting vehicles	1.129 (n/a)	1	4.0	2.655 (0.9)	5

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.759 (1.8)	399	4.808 (1.9)	253	6.743 (1.8)	35
Excluding exiting vehicles	4.932 (2.0)	370	4.949 (2.1)	232	7.028 (2.0)	33

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 19. Data summary sheet for Columbus southeastbound approach

Site	Columbus		
ID	COL01-SEB		
Approach	Southeastbound		
Intersection	Blackmon Rd./Warm Springs Rd.		
County	Muscogee		
City	Columbus		
GDOT District	3		
AADT	n/a		
Date of data collection	Friday, November 02, 2012		
Time of data collection	4:17 PM – 5:56 PM		
Video duration	1:39:00		
Queuing periods at least 1 minute long	1		
Total number of queued minutes	1		
	Total data	vph data	
Number of entering vehicles	544	330	
Number of circulating vehicles	489	297	
Number of exiting vehicles	691	419	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	6.815 (4.6)	99	4.989 (3.5)	184	3.039 (1.2)	141	2.196 (1.2)	189
Excluding exiting vehicles	10.673 (6.9)	30	8.805 (7.9)	112	3.187 (1.3)	60	2.220 (1.6)	132

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	3.271 (1.1)	5	3.6	2.972 (0.8)	107
Excluding exiting vehicles	3.701 (1.4)	6	4.0	3.375 (1.0)	158

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.851 (1.9)	429	4.538 (1.9)	135	3.833 (1.1)	8
Excluding exiting vehicles	6.145 (2.1)	222	4.382 (2.1)	16	4.318 (1.0)	6

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation
[1]Follow up headway observations during all user defined queuing periods > 1 minute
[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts
[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)
[4]Observations that include a rejected gap
[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 20. Data summary sheet for Columbus southwestbound approach

Site	Columbus		
ID	COL01-SWB		
Approach	Southwestbound		
Intersection	Blackmon Rd./Warm Springs Rd.		
County	Muscogee		
City	Columbus		
GDOT District	3		
AADT	n/a		
Date of data collection	Friday, November 02, 2012		
Time of data collection	3:50 PM – 5:57 PM		
Video duration	2:07:00		
Queuing periods at least 1 minute long	3		
Total number of queued minutes	3		
	Total data	vph data	
Number of entering vehicles	914	432	
Number of circulating vehicles	352	167	
Number of exiting vehicles	746	353	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	5.556 (2.3)	115	4.862 (3.6)	310	3.055 (1.1)	89	2.191 (1.2)	221
Excluding exiting vehicles	8.161 (4.6)	25	7.532 (6.4)	130	2.630 (0.7)	27	2.090 (1.6)	105

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	3.119 (0.7)	12	3.6	2.934 (0.7)	264
Excluding exiting vehicles	3.556 (1.3)	22	4.0	3.380 (1.1)	397

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.404 (1.8)	425	4.297 (1.7)	89	4.208 (1.1)	5
Excluding exiting vehicles	5.414 (2.0)	157	3.781 (2.1)	17	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 21. Data summary sheet for Douglasville eastbound approach

Site	Douglasville		
ID	DOU01-EB		
Approach	Eastbound		
Intersection	Duncan Memorial Hwy. (SR 166)/Bill Arp Rd. (SR 5)		
County	Douglas		
City	Douglasville		
GDOT District	7		
AADT	7660		
Date of data collection	Monday, May 14, 2012		
Time of data collection	4:37 PM – 6:54PM		
Video duration	2:17:00		
Queuing periods at least 1 minute long	0		
Total number of queued minutes	0		
	Total data	vph data	
Number of entering vehicles	482	212	
Number of circulating vehicles	450	198	
Number of exiting vehicles	915	401	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	6.610 (5.8)	82	5.904 (4.8)	149	2.629 (1.3)	92	1.823 (1.0)	139
Excluding exiting vehicles	12.359 (6.2)	19	11.866 (10.2)	143	2.639 (1.0)	34	2.397 (1.9)	78

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	n/a (n/a)	0	3.6	2.743 (0.7)	89
Excluding exiting vehicles	n/a (n/a)	0	4.0	2.810 (0.7)	94

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.096 (1.9)	313	3.520 (1.7)	78	n/a (n/a)	0
Excluding exiting vehicles	6.367 (2.3)	131	4.876 (2.6)	12	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation
[1]Follow up headway observations during all user defined queuing periods > 1 minute
[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts
[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)
[4]Observations that include a rejected gap
[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 22. Data summary sheet for Douglasville westbound approach

Site	Douglasville		
ID	DOU01-WB		
Approach	Westbound		
Intersection	Duncan Memorial Hwy. (SR 166)/Bill Arp Rd. (SR 5)		
County	Douglas		
City	Douglasville		
GDOT District	7		
AADT	7660		
Date of data collection	Monday, May 14, 2012		
Time of data collection	4:30 PM – 6:40PM		
Video duration	2:10:00		
Queuing periods at least 1 minute long	1		
Total number of queued minutes	1		
	Total data	vph data	
Number of entering vehicles	734	339	
Number of circulating vehicles	451	209	
Number of exiting vehicles	292	135	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	7.082 (5.2)	36	7.854 (6.2)	226	2.457 (1.1)	58	1.617 (1.1)	121
Excluding exiting vehicles	10.855 (7.3)	15	10.953 (8.4)	167	2.637 (1.1)	43	1.591 (1.1)	107

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	3.167 (0.7)	11	3.6	2.708 (0.7)	217
Excluding exiting vehicles	3.481 (0.9)	16	4.0	2.931 (0.8)	258

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.549 (1.8)	215	3.974 (1.9)	29	n/a (n/a)	0
Excluding exiting vehicles	5.290 (1.8)	162	4.980 (2.0)	10	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation
[1]Follow up headway observations during all user defined queuing periods > 1 minute
[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts
[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)
[4]Observations that include a rejected gap
[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 23. Data summary sheet for Douglasville southbound approach

Site	Douglasville		
ID	DOU01-SB		
Approach	Southbound		
Intersection	Duncan Memorial Hwy. (SR 166)/Bill Arp Rd. (SR 5)		
County	Douglas		
City	Douglasville		
GDOT District	7		
AADT	7660		
Date of data collection	Tuesday, November 1, 2012		
Time of data collection	7:05 AM – 8:32 AM		
Video duration	1:27:00		
Queuing periods at least 1 minute long	0		
Total number of queued minutes	0		
	Total data	vph data	
Number of entering vehicles	200	138	
Number of circulating vehicles	216	149	
Number of exiting vehicles	343	237	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	8.609 (5.9)	28	8.847 (7.7)	93	2.723 (0.8)	17	2.070 (1.1)	46
Excluding exiting vehicles	17.500 (14.2)	8	16.467 (15.4)	68	3.098 (2.0)	5	2.003 (1.8)	21

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	n/a (n/a)	0	3.6	2.689 (1.0)	45
Excluding exiting vehicles	n/a (n/a)	0	4.0	3.108 (1.1)	54

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.010 (2.2)	91	3.733 (1.6)	18	n/a (n/a)	0
Excluding exiting vehicles	6.023 (2.7)	34	n/a (n/a)	0	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 24. Data summary sheet for Emory southeastbound approach

Site	Emory		
ID	EMO01-SEB		
Approach	Southeastbound		
Intersection	N. Decatur Rd./Oxford Rd. NE		
County	Dekalb		
City	Atlanta		
GDOT District	7		
AADT	n/a		
Date of data collection	Friday, October 19, 2012		
Time of data collection	4:10 PM – 5:59 PM		
Video duration	1:49:00		
Queuing periods at least 1 minute long	18		
Total number of queued minutes	32		
	Total data	vph data	
Number of entering vehicles	524	291	
Number of circulating vehicles	1371	755	
Number of exiting vehicles	314	173	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	5.643 (2.7)	160	5.119 (3.5)	107	2.908 (1.0)	604	1.890 (1.2)	287
Excluding exiting vehicles	6.595 (3.0)	117	5.842 (4.3)	95	3.012 (1.2)	496	1.942 (1.4)	264

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	3.680 (1.4)	70	3.6	3.235 (0.6)	101
Excluding exiting vehicles	3.897 (1.5)	85	4.0	3.512 (0.9)	133

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.961 (1.5)	1051	4.834 (1.4)	423	5.241 (1.4)	224
Excluding exiting vehicles	5.555 (1.7)	877	5.175 (1.7)	284	5.134 (1.5)	157

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation
[1]Follow up headway observations during all user defined queuing periods > 1 minute
[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts
[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)
[4]Observations that include a rejected gap
[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 25. Data summary sheet for Fayetteville eastbound approach

Site	Fayetteville		
ID	FAY01-EB		
Approach	Eastbound		
Intersection	Grady Ave./Beauregard Blvd.		
County	Fayette		
City	Fayetteville		
GDOT District	3		
AADT	7920		
Date of data collection	Wednesday, April 11, 2012		
Time of data collection	4:37 PM – 6:49 PM		
Video duration	2:12:00		
Queuing periods at least 1 minute long	3		
Total number of queued minutes	3		
	Total data	vph data	
Number of entering vehicles	693	315	
Number of circulating vehicles	592	270	
Number of exiting vehicles	641	292	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	7.814 (8.2)	262	5.300 (4.4)	262	2.980 (1.0)	143	1.885 (1.1)	182
Excluding exiting vehicles	8.505 (4.3)	36	8.237 (8.9)	155	3.022 (1.4)	86	1.963 (1.8)	124

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	2.840 (1.1)	17	3.6	2.969 (1.0)	188
Excluding exiting vehicles	3.226 (1.5)	24	4.0	3.259 (1.3)	259

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.613 (1.9)	414	4.230 (1.6)	100	3.956 (0.7)	16
Excluding exiting vehicles	6.108 (2.2)	246	4.938 (2.4)	33	3.814 (0.9)	7

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

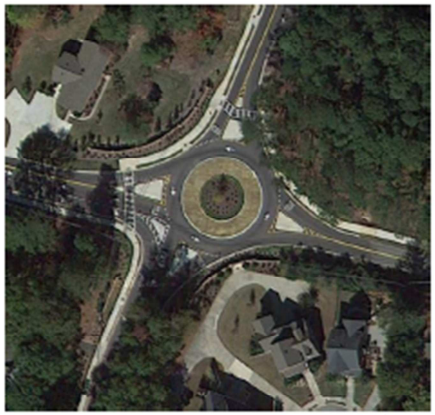
[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 26. Data summary sheet for Fayetteville southbound approach

Site	Fayetteville		
ID	FAY01-SB		
Approach	Southbound		
Intersection	Grady Ave./Beauregard Blvd.		
County	Fayette		
City	Fayetteville		
GDOT District	3		
AADT	6650		
Date of data collection	Wednesday, April 11, 2012		
Time of data collection	4:31 PM – 6:33 PM		
Video duration	2:02:00		
Queuing periods at least 1 minute long	3		
Total number of queued minutes	3		
	Total data	vph data	
Number of entering vehicles	590	291	
Number of circulating vehicles	572	282	
Number of exiting vehicles	199	98	



Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	7.393 (3.8)	48	7.820 (8.2)	156	3.014 (1.4)	94	2.075 (1.2)	129
Excluding exiting vehicles	9.309 (6.0)	32	9.878 (9.8)	125	2.973 (1.2)	73	2.109 (1.5)	111

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	3.288 (1.9)	23	3.6	2.909 (0.9)	213
Excluding exiting vehicles	3.361 (1.8)	25	4.0	3.078 (1.0)	244

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	5.260 (2.0)	271	4.363 (2.2)	43	3.708 (1.8)	5
Excluding exiting vehicles	5.664 (2.0)	216	4.974 (2.3)	25	4.136 (2.0)	3

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 27. Data summary sheet for Fayetteville northbound approach

Site	Fayetteville		
ID	FAY01-NB		
Approach	Northbound		
Intersection	Grady Ave./Beauregard Blvd.		
County	Fayette		
City	Fayetteville		
GDOT District	3		
AADT	6650		
Date of data collection	Wednesday, April 11, 2012		
Time of data collection	7:40 AM – 8:46 AM		
Video duration	1:06:00		
Queuing periods at least 1 minute long	2		
Total number of queued minutes	2		
	Total data	vph data	
Number of entering vehicles	712	648	
Number of circulating vehicles	537	489	
Number of exiting vehicles	772	702	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	6.315 (4.1)	102	5.421 (4.1)	283	3.034 (1.1)	139	2.101 (1.2)	196
Excluding exiting vehicles	8.486 (1.3)	25	8.420 (8.0)	148	3.064 (1.8)	79	1.755 (1.3)	117

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	4.190 (0.4)	8	3.6	3.051 (0.8)	170
Excluding exiting vehicles	4.027 (0.7)	14	4.0	3.414 (1.1)	248

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.590 (1.8)	437	4.739 (1.8)	116	7.211 (2.1)	14
Excluding exiting vehicles	6.327 (2.1)	221	5.291 (2.4)	21	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 28. Data summary sheet for Duluth eastbound approach

Site	Duluth		
ID	DUL01-EB		
Approach	Eastbound		
Intersection	McClure Bridge Rd./W. Lawrenceville St./Irvindale Rd. NW		
County	Gwinnett		
City	Duluth		
GDOT District	3		
AADT	11340		
Date of data collection	Friday, June 1, 2012		
Time of data collection	6:56 AM – 8:56 AM		
Video duration	2:00:00		
Queuing periods at least 1 minute long	0		
Total number of queued minutes	0		
	Total data	vph data	
Number of entering vehicles	455	228	
Number of circulating vehicles	71	36	
Number of exiting vehicles	697	349	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	7.853 (4.7)	37	8.660 (6.3)	192	4.318 (0.6)	23	3.465 (1.2)	67
Excluding exiting vehicles	19.184 (n/a)	1	19.950 (17.3)	45	n/a (n/a)	0	2.648 (1.9)	12

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	n/a (n/a)	0	3.6	2.780 (0.9)	107
Excluding exiting vehicles	n/a (n/a)	0	4.0	3.088 (1.0)	146

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	5.478 (1.7)	127	5.322 (1.3)	18	n/a (n/a)	0
Excluding exiting vehicles	7.852 (2.3)	13	n/a (n/a)	0	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 29. Data summary sheet for Hinesville westbound approach

Site	Hinesville		
ID	HIN01-WB		
Approach	Westbound		
Intersection	Memorial Dr./N. Main St.		
County	Liberty		
City	Hinesville		
GDOT District	5		
AADT	2030		
Date of data collection	Friday, July 27, 2012		
Time of data collection	3:30 PM – 4:41 PM		
Video duration	1:11:00		
Queuing periods at least 1 minute long	0		
Total number of queued minutes	0		
	Total data	vph data	
Number of entering vehicles	442	374	
Number of circulating vehicles	234	198	
Number of exiting vehicles	148	126	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	9.335 (7.4)	19	12.036 (8.8)	61	3.085 (1.0)	20	2.602 (1.5)	32
Excluding exiting vehicles	13.605 (8.3)	13	16.584 (12.6)	51	3.140 (1.1)	11	2.752 (1.9)	27

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	n/a (n/a)	0	3.6	3.120 (0.4)	10
Excluding exiting vehicles	n/a (n/a)	0	4.0	3.635 (0.9)	18

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	5.149 (2.1)	71	3.957 (2.2)	21	n/a (n/a)	0
Excluding exiting vehicles	6.204 (2.5)	51	5.244 (2.5)	13	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 30. Data summary sheet for Hinesville southbound approach

Site	Hinesville		
ID	HIN01-SB		
Approach	Southbound		
Intersection	Memorial Dr./N. Main St.		
County	Liberty		
City	Hinesville		
GDOT District	5		
AADT	5090		
Date of data collection	Friday, July 27, 2012		
Time of data collection	3:38 PM – 5:59 PM		
Video duration	2:21:00		
Queuing periods at least 1 minute long	0		
Total number of queued minutes	0		
	Total data	vph data	
Number of entering vehicles	402	172	
Number of circulating vehicles	247	106	
Number of exiting vehicles	282	120	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	9.200 (6.4)	23	9.493 (7.7)	153	3.195 (1.0)	28	2.256 (1.2)	72
Excluding exiting vehicles	15.938 (16.0)	6	12.975 (9.6)	104	3.512 (1.0)	12	2.444 (1.9)	47

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	n/a (n/a)	0	3.6	2.933 (0.6)	84
Excluding exiting vehicles	n/a (n/a)	0	4.0	3.317 (1.0)	120

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	5.258 (2.0)	123	5.528 (2.1)	14	n/a (n/a)	0
Excluding exiting vehicles	7.111 (2.1)	65	6.339 (1.2)	3	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 31. Data summary sheet for Holly Springs eastbound approach

Site	Holly Springs		
ID	HOL01-EB		
Approach	Eastbound		
Intersection	Holly Springs Rd./Davis Rd.		
County	Cobb		
City	Marietta		
GDOT District	7		
AADT	n/a		
Date of data collection	Tuesday, October 11, 2012		
Time of data collection	4:07 PM – 6:05 PM		
Video duration	1:58:00		
Queuing periods at least 1 minute long	0		
Total number of queued minutes	0		
	Total data	vph data	
Number of entering vehicles	243	124	
Number of circulating vehicles	759	386	
Number of exiting vehicles	185	95	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	7.336 (4.0)	32	9.133 (8.0)	120	2.822 (0.8)	58	1.608 (1.1)	56
Excluding exiting vehicles	9.945 (5.9)	26	11.474 (10.6)	107	3.042 (1.2)	49	1.584 (1.1)	50

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	n/a (n/a)	0	3.6	2.961 (0.5)	36
Excluding exiting vehicles	n/a (n/a)	0	4.0	3.107 (0.6)	51

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.522 (2.0)	146	4.434 (1.8)	56	n/a (n/a)	0
Excluding exiting vehicles	5.183 (2.2)	125	4.876 (2.0)	39	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 32. Data summary sheet for Holly Springs northbound approach

Site	Holly Springs		
ID	HOL01-NB		
Approach	Northbound		
Intersection	Holly Springs Rd./Davis Rd.		
County	Cobb		
City	Marietta		
GDOT District	7		
AADT	n/a		
Date of data collection	Tuesday, October 11, 2012		
Time of data collection	4:17 PM – 6:06 PM		
Video duration	1:49:00		
Queuing periods at least 1 minute long	1		
Total number of queued minutes	1		
	Total data	vph data	
Number of entering vehicles	275	152	
Number of circulating vehicles	1074	592	
Number of exiting vehicles	394	217	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	6.356 (3.2)	90	8.259 (6.7)	105	2.753 (1.1)	231	1.728 (1.2)	123
Excluding exiting vehicles	7.114 (4.4)	70	9.514 (8.6)	95	2.600 (0.9)	175	1.667 (1.2)	117

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	4.417 (n/a)	1	3.6	2.947 (0.7)	29
Excluding exiting vehicles	3.882 (0.7)	3	4.0	3.137 (0.7)	42

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.727 (1.8)	444	4.638 (1.7)	249	4.111 (1.0)	7
Excluding exiting vehicles	4.621 (1.9)	362	4.487 (1.8)	176	3.038 (1.4)	4

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 33. Data summary sheet for Villa Rica southwestbound approach

Site	Villa Rica		
ID	VIL01-SWB		
Approach	Southwestbound		
Intersection	Villa Rica Rd. SW/W. Sandtown Rd. SW		
County	Cobb		
City	Marietta		
GDOT District	7		
AADT	n/a		
Date of data collection	Tuesday, March 27, 2012		
Time of data collection	4:22 PM – 6:46 PM		
Video duration	2:24:00		
Queuing periods at least 1 minute long	8		
Total number of queued minutes	8		
	Total data	vph data	
Number of entering vehicles	1271	530	
Number of circulating vehicles	333	139	
Number of exiting vehicles	676	282	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	4.435 (2.3)	73	3.551 (2.9)	345	2.769 (0.8)	90	1.738 (1.0)	267
Excluding exiting vehicles	9.903 (8.8)	12	5.666 (4.6)	116	3.088 (1.4)	39	1.718 (1.4)	133

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	2.535 (0.6)	81	3.6	2.593 (0.6)	571
Excluding exiting vehicles	2.812 (1.0)	133	4.0	2.878 (0.9)	825

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.038 (1.4)	430	3.384 (1.2)	50	4.001 (1.0)	7
Excluding exiting vehicles	6.667 (1.9)	184	4.521 (1.6)	6	5.333 (0.7)	2

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation
[1]Follow up headway observations during all user defined queuing periods > 1 minute
[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts
[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)
[4]Observations that include a rejected gap
[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 34. Data summary sheet for Newnan eastbound approach

Site	Newnan		
ID	NEW01-EB		
Approach	Eastbound		
Intersection	E. Broad St./E. Newnan Rd.		
County	Coweta		
City	Newnan		
GDOT District	3		
AADT	9790		
Date of data collection	Thursday, October 25, 2012		
Time of data collection	7:33 AM – 9:28 AM		
Video duration	1:55:00		
Queuing periods at least 1 minute long	1		
Total number of queued minutes	1		
	Total data	vph data	
Number of entering vehicles	536	280	
Number of circulating vehicles	388	203	
Number of exiting vehicles	665	347	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	6.464 (6.4)	58	4.282 (3.8)	252	2.837 (1.5)	68	1.676 (1.4)	89
Excluding exiting vehicles	7.162 (2.8)	22	7.720 (7.8)	146	3.247 (2.3)	37	1.609 (1.8)	58

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	4.077 (2.6)	9	3.6	3.037 (0.8)	120
Excluding exiting vehicles	3.892 (2.5)	10	4.0	3.167 (0.9)	172

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.647 (2.1)	215	5.045 (2.1)	64	n/a (n/a)	0
Excluding exiting vehicles	5.974 (2.5)	117	4.883 (2.6)	9	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 35. Data summary sheet for Newnan westbound approach

Site	Newnan		
ID	NEW01-WB		
Approach	Westbound		
Intersection	E. Broad St./E. Newnan Rd.		
County	Coweta		
City	Newnan		
GDOT District	3		
AADT	9790		
Date of data collection	Thursday, October 25, 2012		
Time of data collection	7:35 AM – 9:27 AM		
Video duration	1:52:00		
Queuing periods at least 1 minute long	6		
Total number of queued minutes	6		
	Total data	vph data	
Number of entering vehicles	690	370	
Number of circulating vehicles	461	247	
Number of exiting vehicles	547	294	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	6.572 (3.8)	86	4.855 (4.4)	181	3.080 (1.0)	111	2.436 (1.2)	224
Excluding exiting vehicles	10.742 (9.2)	25	7.752 (8.0)	116	2.774 (0.9)	60	2.128 (9.2)	139

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	2.927 (0.8)	32	3.6	3.113 (0.9)	252
Excluding exiting vehicles	3.242 (0.9)	52	4.0	3.409 (1.2)	368

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	4.711 (1.7)	421	4.276 (1.8)	66	4.138 (1.3)	12
Excluding exiting vehicles	5.587 (1.8)	221	4.538 (2.2)	16	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 36. Data summary sheet for Roswell eastbound approach (05/15/12)

Site	Roswell		
ID	ROS01-EB		
Approach	Eastbound		
Intersection	Grimes Bridge Rd./Norcross St./Warsaw Rd./Medlody Ln.		
County	Fulton		
City	Roswell		
GDOT District	7		
AADT	12440		
Date of data collection	Tuesday, May 15, 2012		
Time of data collection	4:27 PM – 6:33 PM		
Video duration	2:06:00		
Queuing periods at least 1 minute long	7		
Total number of queued minutes	8		
	Total data	vph data	
Number of entering vehicles	933	445	
Number of circulating vehicles	373	178	
Number of exiting vehicles	1732	825	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	3.734 (1.4)	208	3.155 (2.4)	358	2.413 (0.9)	200	1.784 (0.7)	376
Excluding exiting vehicles	6.503 (3.1)	10	9.876 (8.3)	132	2.918 (1.2)	27	2.063 (1.5)	130

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	2.956 (0.8)	35	3.6	2.696 (0.7)	191
Excluding exiting vehicles	3.624 (1.5)	102	4.0	3.327 (1.1)	505

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	3.313 (1.2)	784	3.389 (1.2)	179	3.655 (0.9)	35
Excluding exiting vehicles	6.417 (1.6)	167	3.736 (1.3)	4	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 37. Data summary sheet for Roswell southwestbound approach (5/15/12)

Site	Roswell		
ID	ROS01-SWB		
Approach	Southwestbound		
Intersection	Grimes Bridge Rd./Norcross St./Warsaw Rd./Medlody Ln.		
County	Fulton		
City	Roswell		
GDOT District	7		
AADT	n/a		
Date of data collection	Tuesday, May 15, 2012		
Time of data collection	4:17 PM – 6:24 PM		
Video duration	2:07:00		
Queuing periods at least 1 minute long	15		
Total number of queued minutes	17		
	Total data	vph data	
Number of entering vehicles	987	467	
Number of circulating vehicles	1118	529	
Number of exiting vehicles	417	198	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	5.120 (2.2)	179	3.394 (3.1)	250	2.696 (0.9)	202	1.715 (1.0)	324
Excluding exiting vehicles	6.243 (4.3)	123	4.103 (4.7)	184	2.632 (0.9)	139	1.641 (1.1)	274

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	2.725 (0.7)	94	3.6	2.630 (0.7)	378
Excluding exiting vehicles	3.190 (1.5)	132	4.0	2.913 (1.0)	474

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	3.926 (1.6)	705	3.817 (1.5)	191	3.906 (1.3)	78
Excluding exiting vehicles	4.138 (1.8)	538	4.230 (1.7)	104	4.530 (1.6)	38

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation
[1]Follow up headway observations during all user defined queuing periods > 1 minute
[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts
[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)
[4]Observations that include a rejected gap
[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 38. Data summary sheet for Roswell eastbound approach (10/23/12)

Site	Roswell		
ID	ROS02-EB		
Approach	Southwestbound		
Intersection	Grimes Bridge Rd./Norcross St./Warsaw Rd./Medlody Ln.		
County	Fulton		
City	Roswell		
GDOT District	7		
AADT	12440		
Date of data collection	Tuesday, October 23, 2012		
Time of data collection	3:45 PM – 5:13 PM		
Video duration	1:28:00		
Queuing periods at least 1 minute long	6		
Total number of queued minutes	7		
	Total data	vph data	
Number of entering vehicles	710	485	
Number of circulating vehicles	276	189	
Number of exiting vehicles	1273	868	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	4.094 (1.2)	193	2.968 (1.5)	273	2.592 (0.8)	180	2.067 (1.0)	291
Excluding exiting vehicles	8.674 (6.4)	15	6.083 (5.7)	95	2.647 (0.8)	24	2.062 (1.6)	113

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	2.883 (1.0)	32	3.6	2.684 (0.6)	125
Excluding exiting vehicles	3.747 (1.7)	92	4.0	3.447 (1.2)	352

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	3.633 (1.3)	664	3.338 (1.2)	203	3.227 (1.2)	28
Excluding exiting vehicles	6.234 (1.9)	152	4.557 (2.8)	2	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 39. Data summary sheet for Roswell southwestbound approach (10/23/12)

Site	Roswell		
ID	ROS02-SWB		
Approach	Southwestbound		
Intersection	Grimes Bridge Rd./Norcross St./Warsaw Rd./Medlody Ln.		
County	Fulton		
City	Roswell		
GDOT District	7		
AADT	n/a		
Date of data collection	Tuesday, October 23, 2012		
Time of data collection	3:37 PM – 5:16 PM		
Video duration	1:39:00		
Queuing periods at least 1 minute long	30		
Total number of queued minutes	43		
	Total data	vph data	
Number of entering vehicles	917	556	
Number of circulating vehicles	433	263	
Number of exiting vehicles	1166	707	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	4.395 (1.7)	199	3.626 (3.2)	214	2.732 (0.8)	259	1.593 (1.0)	391
Excluding exiting vehicles	5.101 (2.1)	134	4.841 (4.3)	161	2.794 (1.1)	187	1.620 (1.2)	339

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	2.926 (0.8)	248	3.6	2.762 (0.6)	267
Excluding exiting vehicles	3.249 (1.2)	336	4.0	3.061 (1.0)	351

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	3.863 (1.5)	849	3.780 (1.3)	217	3.648 (1.1)	132
Excluding exiting vehicles	4.443 (1.7)	660	4.064 (1.6)	112	4.032 (1.5)	58

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation
[1]Follow up headway observations during all user defined queuing periods > 1 minute
[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts
[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)
[4]Observations that include a rejected gap
[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 40. Data summary sheet for St. Simons westbound approach

Site	St. Simons		
ID	STS01-WB		
Approach	Westbound		
Intersection	Lawrence Rd./Frederica Rd.		
County	Glynn		
City	St. Simons Island		
GDOT District	5		
AADT	n/a		
Date of data collection	Saturday, July 28, 2012		
Time of data collection	2:20 PM – 4:26 PM		
Video duration	2:06:00		
Queuing periods at least 1 minute long	0		
Total number of queued minutes	0		
	Total data	vph data	
Number of entering vehicles	236	113	
Number of circulating vehicles	332	159	
Number of exiting vehicles	245	117	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	20.598 (13.3)	14	17.056 (14.9)	129	3.032 (0.9)	10	2.080 (1.6)	20
Excluding exiting vehicles	28.966 (26.5)	9	23.041 (19.6)	110	3.787 (2.1)	6	1.909 (1.8)	14

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	n/a (n/a)	0	3.6	2.543 (0.6)	24
Excluding exiting vehicles	n/a (n/a)	0	4.0	2.843 (0.9)	30

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	5.587 (2.7)	44	5.517 (2.5)	15	n/a (n/a)	0
Excluding exiting vehicles	6.567 (3.0)	29	7.733 (2.6)	9	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

Table 41. Data summary sheet for St. Simons eastbound approach

Site	St. Simons		
ID	STS01-EB		
Approach	Eastbound		
Intersection	Lawrence Rd./Frederica Rd.		
County	Glynn		
City	St. Simons Island		
GDOT District	5		
AADT	n/a		
Date of data collection	Saturday, July 28, 2012		
Time of data collection	2:23 PM – 3:32 PM		
Video duration	1:09:00		
Queuing periods at least 1 minute long	0		
Total number of queued minutes	0		
	Total data	vph data	
Number of entering vehicles	273	238	
Number of circulating vehicles	186	162	
Number of exiting vehicles	261	227	

Source: Google Earth™, accessed 8/16/2013

Gap/Lag Data	**Accepted gaps**		**Accepted lags**		**Rejected gaps**		**Rejected lags**	
	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n	avg. (std. dev.) (s)	n
Including exiting vehicles	17.255 (11.9)	9	13.596 (11.5)	144	3.144 (1.5)	9	1.843 (1.1)	18
Excluding exiting vehicles	29.176 (13.7)	4	21.459 (17.7)	97	3.224 (1.7)	6	1.866 (1.5)	9

Follow-up Headway	**Queued Data**		**Move-up Data**		
	t_f (std. dev.) (s)	n	Move-up time (s)	t_f (std. dev.) (s)	n
Including exiting vehicles	n/a (n/a)	0	3.6	2.321 (0.9)	33
Excluding exiting vehicles	n/a (n/a)	0	4.0	2.519 (1.0)	37

Critical Headway	**NCHRP Method 1**		**NCHRP Method 2**		**NCHRP Method 3**	
	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n	t_c (std. dev.) (s)	n
Including exiting vehicles	5.431 (2.5)	36	6.030 (2.9)	9	n/a (n/a)	0
Excluding exiting vehicles	7.079 (2.7)	19	n/a (n/a)	0	n/a (n/a)	0

Legend: avg. = average; n = number of observations; tc = critical headway; std. dev. = standard deviation

[1]Follow up headway observations during all user defined queuing periods > 1 minute

[2]Follow up headway observations determined using move-up time thresholds from > 1 minute user defined queuing periods of all roundabouts

[3]Observations of gap acceptance (accepted/rejected gaps and rejected lags)

[4]Observations that include a rejected gap

[5]Observations that include a rejected gap and occur during user defined queuing periods > 1 minute

APPENDIX F: ROUNDABOUT RESULTS EXITING VEHICLES

F.1 CRITICAL HEADWAYS

As for the “without exiting vehicles” analysis, critical headway values for the “with exiting vehicles” analysis were determined using each of the three NCHRP critical headway methods at each of the data collection sites. For critical headway analysis including exiting vehicles, the “Wisconsin projected travel time” method described in section 2.6.1 was used in projecting the exiting vehicles forward to the conflict point. The projected travel times and critical headway value for each site using these methods are provided in Table 42 and Table 43 respectively. Table 44 shows the average, weighted average, and median critical headway values across all sites.

As was the case with the “without exiting vehicles” analysis, the critical headway values used to consider the impact of exiting vehicles used in this study’s recommended capacity equations were calculated using NCHRP’s critical headway Method 2. The weighted average critical headway values of 4.192 and 4.747 seconds were used in the calibrated equations for “with exiting vehicles” and “without exiting vehicles” respectively. For the “with exiting vehicles” case, the average critical headway values for all sites as determined by NCHRP Methods 1, and 3 were 4.277 and 4.270 seconds respectively which is very similar to the recommended value of 4.192 seconds.

Table 42. Projected travel time measured for roundabout sites

Site	Projected Travel Time (seconds)
ALP01-SB	0.902
COV01-SB	1.549
COV01-NB	1.493
COV01-WB	2.323
COV01-EB	1.244
COL01-SEB	1.374
COL01-SWB	1.309
DOU01-EB	0.615
DOU01-WB	0.893
DOU01-SB	1.355
EMO01-SEB	1.292
FAY01-EB	1.290
FAY01-SB	1.463
FAY01-NB	1.481
DUL01-EB	2.229
HIN01-WB	1.317
HIN01-SB	1.161
HOL01-EB	0.250
HOL01-NB	2.263
VIL01-SWB	0.910
NEW01-EB	0.965
NEW01-WB	1.958
ROS01-EB	0.927
ROS01-SWB	1.212
ROS02-EB	1.043
ROS02-SWB	1.291
STS01-WB	1.177
STS01-EB	0.861

Table 43. Critical headway with exiting vehicles

Sites	NCHRP Method 1[1]			NCHRP Method 2[2]			NCHRP Method 3[3]		
	t_c (s)	n	std. dev.	t_c (s)	n (% of NCHRP Method 1)	std. dev.	t_c (s)	n (% of NCHRP Method 1)	std. dev.
ALP01-SB	3.734	879	1.3	3.467	341 (39%)	1.3	2.960	26 (3%)	0.9
COV01-SB	3.943	1208	1.3	3.755	213 (18%)	1.1	3.715	186 (15%)	1.1
COV01-NB	4.109	1406	1.3	4.138	327 (23%)	1.6	4.158	171 (12%)	1.4
COV01-WB	4.800	642	1.9	4.774	222 (35%)	1.7	4.133	11 (2%)	1.3
COV01-EB	4.759	399	1.8	4.808	253 (63%)	1.9	6.743	35 (9%)	1.8
COL01-SEB	4.851	429	1.9	4.538	135 (31%)	1.9	3.833	8 (2%)	1.1
COL01-SWB	4.404	425	1.8	4.297	89 (21%)	1.7	4.208	5 (1%)	1.1
DOU01-EB	4.096	313	1.9	3.520	78 (25%)	1.7	n/a	0 (0%)	n/a
DOU01-WB	4.549	215	1.8	3.974	29 (13%)	1.9	n/a	0 (0%)	n/a
DOU01-SB	4.010	91	2.2	3.733	18 (20%)	1.6	n/a	0 (0%)	n/a
EMO01-SEB	4.961	1051	1.5	4.834	423 (40%)	1.4	5.241	224 (21%)	1.4
FAY01-EB	4.613	414	1.9	4.230	100 (24%)	1.6	3.956	16 (4%)	0.7
FAY01-SB	5.260	271	2.0	4.363	43 (16%)	2.2	3.708	5 (2%)	1.8
FAY01-NB	4.590	437	1.8	4.739	116 (27%)	1.8	7.211	14 (3%)	2.1
DUL01-EB	5.478	127	1.7	5.322	14 (6%)	1.3	n/a	0 (0%)	n/a
HIN01-WB	5.149	71	2.1	3.957	21 (30%)	2.2	n/a	0 (0%)	n/a
HIN01-SB	5.258	123	2.0	5.528	14 (11%)	2.1	n/a	0 (0%)	n/a
HOL01-EB	4.522	146	2.0	4.434	56 (38%)	1.8	n/a	0 (0%)	n/a
HOL01-NB	4.727	444	1.8	4.638	249 (56%)	1.7	4.111	7 (2%)	1.0
VIL01-SWB	4.038	430	1.4	3.384	50 (12%)	1.2	4.001	7 (2%)	1.0
NEW01-EB	4.647	215	2.1	5.045	64 (30%)	2.1	n/a	0 (0%)	n/a
NEW01-WB	4.711	421	1.7	4.276	66 (16%)	1.8	4.138	12 (3%)	1.3
ROS01-EB	3.313	784	1.2	3.389	179 (23%)	1.2	3.655	35 (4%)	0.9
ROS01-SWB	3.926	705	1.6	3.817	191 (27%)	1.5	3.906	78 (11%)	1.3
ROS02-EB	3.633	664	1.3	3.338	203 (31%)	1.2	3.227	28 (4%)	1.2
ROS02-SWB	3.863	849	1.5	3.780	217 (26%)	1.3	3.648	132 (16%)	1.1
STS01-WB	5.587	44	2.7	5.517	15 (34%)	2.5	n/a	0 (0%)	n/a
STS01-EB	5.431	36	2.5	6.030	9 (25%)	2.9	n/a	0 (0%)	n/a
Total		13239			3739 (28%)			1000 (8%)	
Average (weighted)	4.277		2.136	4.192		2.039	4.270		1.315

Legend: n = number of observations; t_c = critical headway; std. dev. = standard deviation
[1]All observations of gap acceptance (accepted/rejected gaps and rejected lags)
[2]Observations that include a rejected gap
[3]Observations that include a rejected gap and occur during user defined queuing periods > 1 min

Table 44. Critical headway values for all sites using NCHRP Report 572 critical headway Method 2

	With exiting, All Sites
Average (s)	4.344
Weighted average (s)	**4.192**
Median (s)	4.230
N	3739

F.2 FOLLOW-UP HEADWAY

Follow-up headway values were determined using both NCHRP Report 572's "queued data" and "move-up" time methods. Move-up time thresholds were established separately for the analysis "with" and "without" exiting vehicles. This is required as the inclusion of exiting vehicles creates a different circulating vehicle gap distribution (essentially dividing larger gaps into smaller gaps) reducing the number of follow-up headway measurement opportunities. Figure 41 displays the frequency of move-up times with exiting vehicles under queued conditions. The 95th percentile move-up time threshold was found to be 3.6 seconds. Table 45 provides follow-up headway values for both the "queued data" and the "move-up time" methods when including exiting vehicles. As for the "without exiting vehicle" analysis the follow-up headway values used in this study's recommended capacity equations were calculated using NCHRP's "move-up time" method. Table 46. shows the average, weighted average, and median follow-up headway values for all sites including exiting vehicles. As with critical gap the weighted average follow-up headway value of 2.788 and 3.265 seconds for data sets with and without exiting vehicles respectively were used for the calibrated equations.

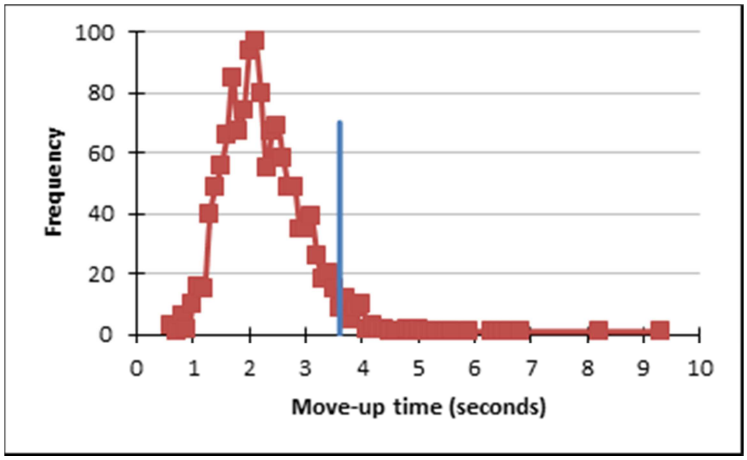


Figure 41. Move-up time frequency for queued data with exiting vehicles (n=1371)

F.3 EXITING VEHICLE COMPARISON

As discussed critical and follow-up headway values were calculated using data "with" and "without" exiting vehicles. Figure 42 compares the critical headway values for each approach determined using NCHRP Report 572 Method 2 when exiting vehicles are included and excluded from the analysis. The sites DOU01-SB, DUL01-EB, and STS01-EB do not have critical headway estimates because the sites did not have the gap data necessary for calculating critical headway "without exiting vehicles" using Method 2.

A majority of the critical headway values for the analysis including exiting vehicles are smaller than the critical headway values excluding exiting vehicles. The smaller headway values are a result of the splitting of a single gap into smaller gaps by the exiting vehicles. Therefore, entering vehicles accept and reject smaller gaps than if exiting vehicles were not included. This also explains why there are 36% more critical headway observations in the analyses including exiting vehicles than the analyses excluding exiting vehicles.

Table 45. Follow-up headway including exiting vehicles

Site	Queued Data			Move-up Time < 3.6 sec.		
	n	t_f (s)	std. dev.	n	t_f (s)	std. dev.
ALP01-SB	1	2.624	n/a	118	2.813	0.7
COV01-SB	514	2.778	0.8	415	2.635	0.6
COV01-NB	170	2.736	0.7	241	2.775	0.7
COV01-WB	7	2.503	1.2	117	2.575	0.7
COV01-EB	1	1.129	n/a	3	2.204	0.9
COL01-SEB	5	3.271	1.1	107	2.972	0.8
COL01-SWB	12	3.119	0.7	264	2.934	0.7
DOU01-EB	0	n/a	n/a	89	2.743	0.7
DOU01-WB	11	3.167	0.7	217	2.708	0.7
DOU01-SB	0	n/a	n/a	45	2.689	1.0
EMO01-SEB	70	3.68	1.4	101	3.235	0.6
FAY01-EB	17	2.84	1.1	188	2.969	1.0
FAY01-SB	23	3.288	1.9	213	2.909	0.9
FAY01-NB	8	4.19	0.4	170	3.051	0.8
DUL01-EB	0	n/a	n/a	107	2.78	0.9
HIN01-WB	0	n/a	n/a	10	3.12	0.4
HIN01-SB	0	n/a	n/a	84	2.933	0.6
HOL01-EB	0	n/a	n/a	36	2.961	0.5
HOL01-NB	1	4.417	n/a	29	2.947	0.7
VIL01-SWB	81	2.535	0.6	571	2.593	0.6
NEW01-EB	9	4.077	2.6	120	3.037	0.8
NEW01-WB	32	2.927	0.8	252	3.113	0.9
ROS01-EB	35	2.956	0.8	191	2.696	0.7
ROS01-SWB	94	2.725	0.7	378	2.63	0.7
ROS02-EB	32	2.883	1.0	125	2.684	0.6
ROS02-SWB	248	2.926	0.8	267	2.762	0.6
STS01-WB	0	n/a	n/a	24	2.543	0.6
STS01-EB	0	n/a	n/a	33	2.321	0.9
Total	1371			4515		
Weighted Average (s)		2.871	0.836		2.788	0.714

Table 46. Average, weighted average, and median follow-up headway including exiting vehicles

	All Sites, With exiting
Average (s)	2.798
Weighted average (s)	**2.788**
Median (s)	2.694
n	4515

Follow-up headway is influenced more by the inclusion of exiting vehicles than is critical headway. Figure 43 compares the follow-up headway values for each approach when exiting vehicles are included to the follow-up headway values when exiting vehicle are excluded in the data analysis. For every site, the follow-up headway value is smaller when exiting vehicles are included than when exiting vehicles are not included. The five sites with the biggest decrease in follow-up headway when exiting vehicles were included are COV01-SB, ROS02-EB, ROS01-EB, COV01-NB, and ALP01-SB. These five sites are five of the top six sites with the highest percentage of exiting vehicles at 78%, 82%, 82%, 84%, and 59% respectively. Therefore, the proportion of conflicting vehicles that are exiting vehicles does impact the corresponding follow-up headway values. Figure 44 displays the percentage of conflicting vehicles that are circulating vehicles and exiting vehicles at each site.

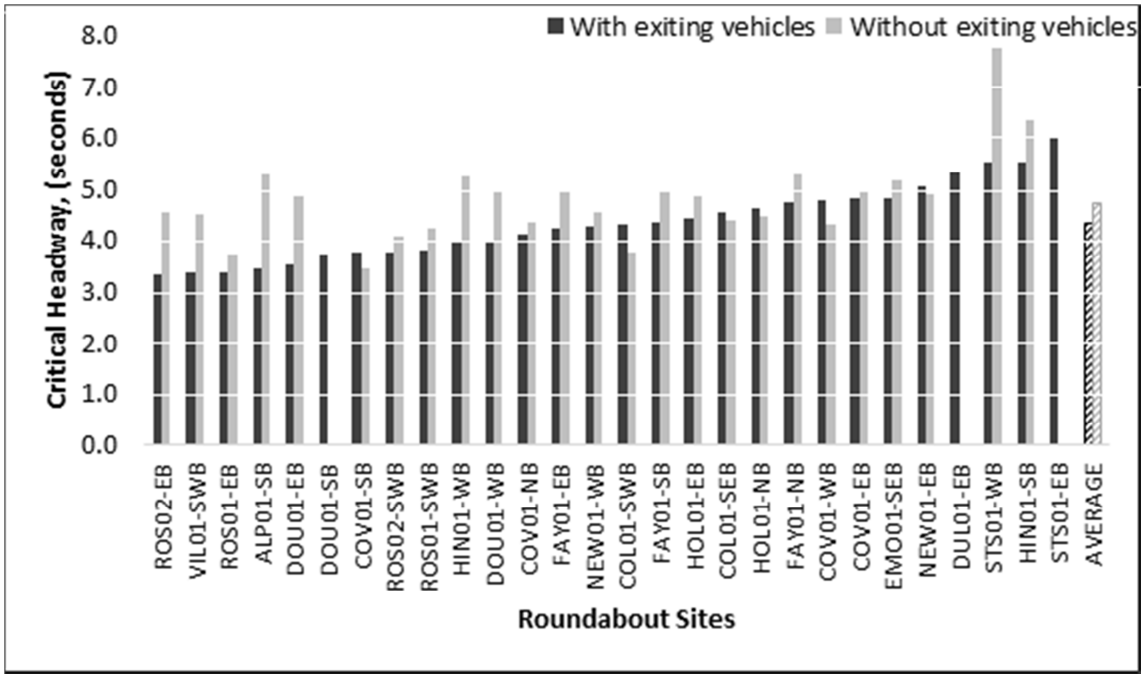


Figure 42. Comparison of critical headway values with and without exiting vehicles by approach

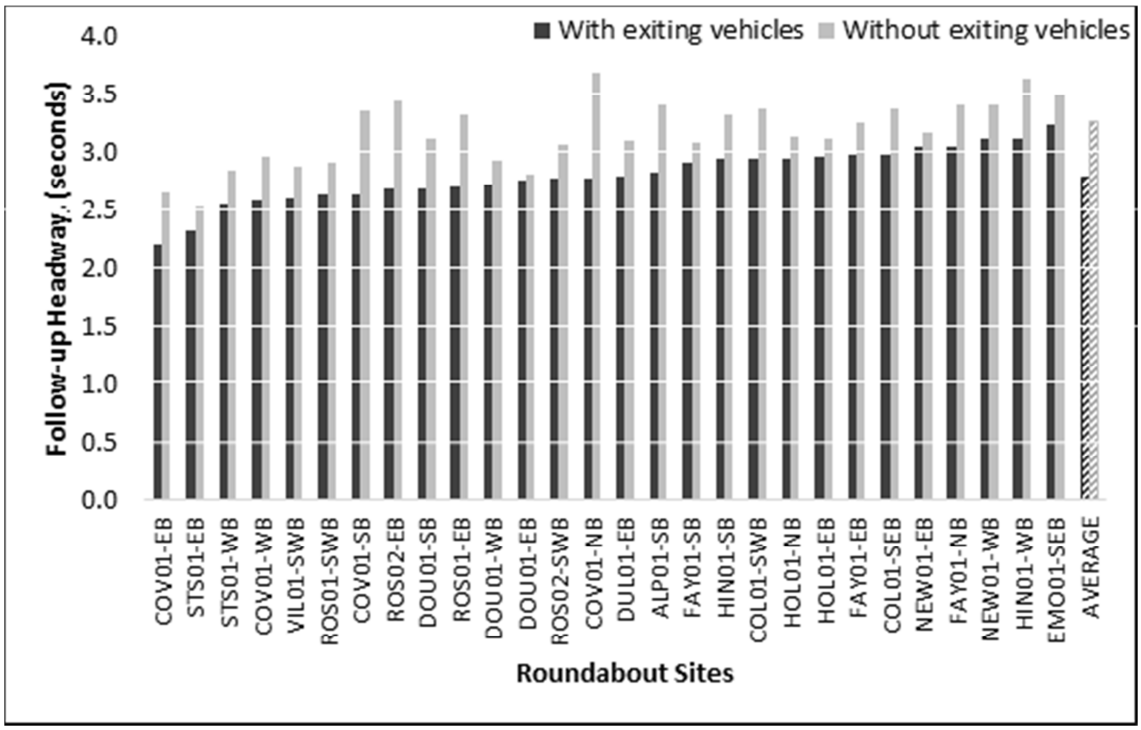


Figure 43. Comparison of follow-up headway values with and without exiting vehicles by approach

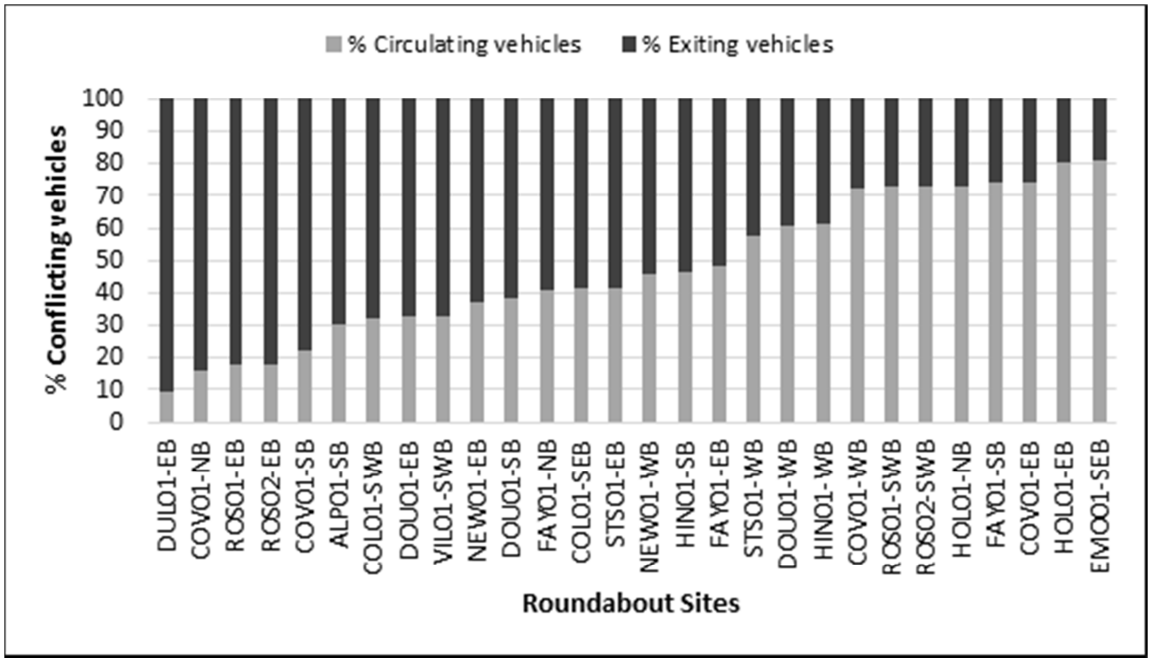


Figure 44. Percentage of conflicting vehicles that are exiting and circulating vehicles

Figure 45 illustrates the calibrated roundabout capacity equations "with exiting vehicles" for all 28 sites. The legend to the right of the graph displays the roundabout sites in order (top to bottom) of highest to lowest entry capacity at the conflicting flow of 1500 vph, the highest conflicting flow included. The dashed line represents the including exiting vehicles calibrated model using the overall weighted average critical and follow-up headway values of 4.192 and 2.788 seconds respectively.

For the analysis including exiting vehicles the calibrated equation is shown below as Equation 17.

$$c_{e,pce} = 1291e^{(-0.0008 \times v_{c,pce})} \quad \textbf{(17)}$$

Where:

$c_{e,pce}$ = capacity of the approach lane under consideration in passenger car equivalents, veh/h

$v_{c,pce}$ = conflicting flow in passenger car equivalents, veh/h

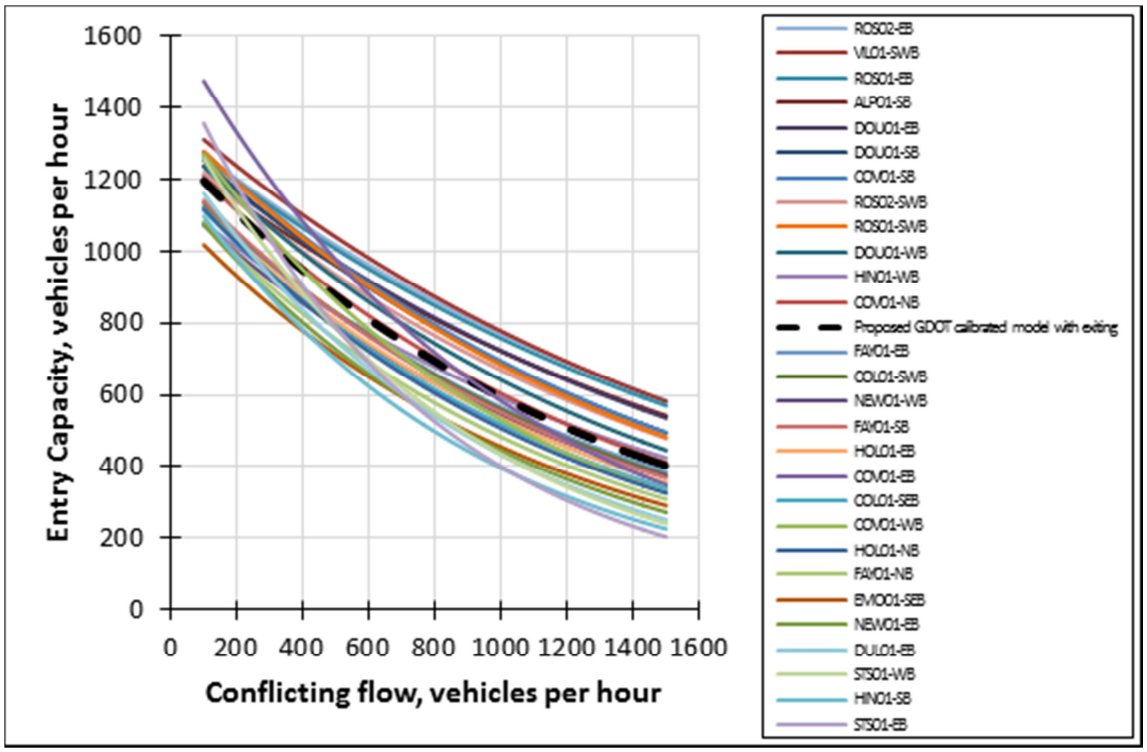


Figure 45. Calibrated single-lane roundabout capacity equations including exiting vehicles by approach

F.4 MODIFIED LIST

Figure 46 displays the calibrated roundabout capacity equations "with exiting vehicles" for the sites on the modified List. The legend to the right of the graph displays the roundabout sites in order (top to bottom) of highest to lowest entry capacity at the conflicting flow of 1500 vph. The dashed line represents the proposed calibrated model using the overall weighted average critical and follow-up headway values of 4.262 and 2.798 seconds respectively.

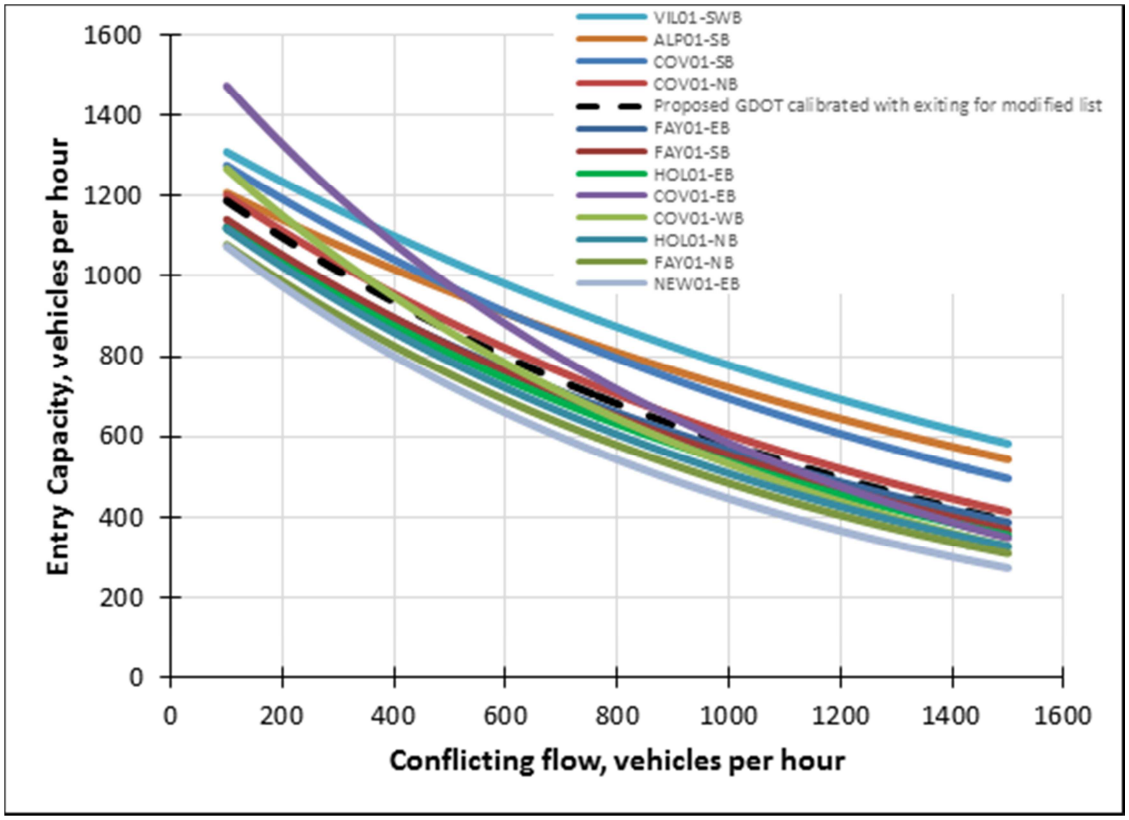


Figure 46. Calibrated single-lane roundabout capacity equations including exiting vehicles for modified list

The weighted average headway values were used to develop capacity equations for analysis including exiting vehicles and excluding exiting vehicles for the modified List. The calibrated equation for the modified List including exiting vehicles is shown below as Equation 18.

$$c_{e,pce} = 1287e^{(-0.0008 \times v_{c,pce})} \quad \textbf{(18)}$$

Where:

$c_{e,pce}$ = capacity of the approach lane under consideration in passenger car equivalents, veh/h

$v_{c,pce}$ = conflicting flow in passenger car equivalents, veh/h

Table 47 and Table 48 show there is an insignificant difference between "all site" locations and the "modified list" for critical and follow-up headways respectively. These results suggest that difference in geometry between the modified list roundabouts and the other roundabouts was not sufficient to affect the operations of the roundabout. Thus, it is recommended to use the calibrated equations based on all 28 roundabout approaches.

Table 47. Critical headway values for modified site list using NCHRP Report 572 critical headway Method 2

	ALL 28 APPROACHES		**MODIFIED LIST (13 APPROACHES)**	
	With exiting	Without exiting	With exiting	Without exiting
Average (s)	4.344	4.445	4.312	4.686
Weighted average (s)	4.192	4.747	4.262	4.738
Median (s)	4.230	4.938	4.276	4.876
Number of observations	3739	1344	2100	758

Table 48. Follow-up headway values for modified site list using NCHRP Report 572 move-up time method

	ALL 28 APPROACHES		**MODIFIED LIST (13 APPROACHES)**	
	With exiting	Without exiting	With exiting	Without exiting
Average (s)	2.798	3.171	2.814	3.194
Weighted average (s)	2.788	3.265	2.798	3.312
Median (s)	2.694	3.040	2.686	3.084
Number of observations	4515	8156	2473	5029