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Determining Louisiana's Roundabout Capacity

by

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13. Abstract

This study calibrated the HCM roundabout capacity models to incorporate local Louisiana driver behavior at single-lane roundabouts, using gap acceptance parameters measured from field data at forty-one approaches from seventeen roundabouts throughout the state. The correlation between these gap acceptance parameters and certain geometric features were explored. Delays, queue lengths, and levels of service (LOS) were determined using appropriate equations from the HCM 6th Edition (HCM 6). Furthermore, a SIDRA analysis was undertaken to determine whether the default environment factor (EF) of 1.2 currently being used by DOTD for the Build Year design was appropriate. Using a maximum likelihood estimation analysis to determine the gap acceptance parameters, it was deduced that the average follow-up time and critical headway for Louisiana drivers is 3.36 seconds and 4.76 seconds respectively. The national averages from the HCM 6 were 2.60 seconds and 4.70 seconds respectively. No notable difference in gap acceptance parameters existed across DOTD districts/parishes, nor different functional classes of roads, although arterials had the least follow-up times and critical headways. To determine whether geometric features contribute to the observed follow-up times, inscribed circle diameter (ICD), splitter width, central island diameter, and approach lane width were compared to the follow-up times. Even though follow-up times most correlated with ICD, none of the relationships proved strong enough to be included in the local model calibration. The local model was developed by recalibrating the HCM 6 model to a fixed intercept based on the average observed follow-up time, and a slope parameter based on regression of the data. The local model results showed that it performed closer to the HCM 2010 model than the HCM 6 model. The SIDRA analysis showed that an environment factor (EF) of 1.06 generated approach capacities with the least root mean squared error (RMSE) when compared to approach volumes from the local model for given circulatory volumes. This value corresponds to an average age of eight years for the 17 roundabouts evaluated. Comparing delays, queue lengths, and LOS theoretically determined for the roundabout approaches, to the SIDRA outputs generated mixed results. Future studies should find innovative ways to determine these parameters directly from the field to compare with SIDRA outputs.

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Each research project will have an advisory committee appointed by the LTRC Director. The Project Review Committee is responsible for assisting the LTRC Administrator or Manager in the development of acceptable research problem statements, requests for proposals, review of research proposals, oversight of approved research projects, and implementation of findings.

LTRC appreciates the dedication of the following Project Review Committee Members in guiding this research study to fruition.

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Abstract

This study calibrated the HCM roundabout capacity models to incorporate local Louisiana driver behavior at single-lane roundabouts, using gap acceptance parameters measured from field data at 41 approaches from 17 roundabouts throughout the state. The correlation between these gap acceptance parameters and certain geometric features were explored. Delays, queue lengths, and levels of service (LOS) were determined using appropriate equations from the HCM 6. Furthermore, a SIDRA INTERSECTION (SIDRA) analysis was undertaken to determine whether the default environment factor (EF) of 1.2 currently being used by DOTD for the Build Year design was appropriate.

Using a maximum likelihood estimation (MLE) analysis to determine the gap acceptance parameters, it was deduced that the average follow-up time and critical headway for Louisiana drivers are 3.36 seconds and 4.76 seconds, respectively. The national averages from the HCM 6 were 2.60 seconds and 4.70 seconds, respectively. No notable difference in gap acceptance parameters existed across DOTD districts/parishes, nor different functional classes of roads, although arterials had the least follow-up times and critical headways. To determine whether geometric features contribute to the observed follow-up times, inscribed circle diameter (ICD), splitter width, central island diameter, and approach lane width were compared to the follow-up times. Even though follow-up times mostly correlated with ICD, none of the relationships proved strong enough to be included in the local model calibration. The local model was developed by recalibrating the HCM 6 model to a fixed intercept based on the average observed follow-up time, and a slope parameter based on regression of the data. The local model results showed that it performed closer to the HCM 2010 model than the HCM 6 model.

The SIDRA analysis showed that an environment factor (EF) of 1.06 generated approach capacities with the least root mean squared error (RMSE) when compared to approach volumes from the local model for given circulatory volumes. This value corresponds to an average age of eight years for the 17 roundabouts evaluated. Comparing delays, queue lengths, and LOS theoretically determined for the roundabout approaches, to the SIDRA outputs generated mixed results. Future studies should find innovative ways to determine these parameters directly from the field to compare with SIDRA outputs.

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Implementation Statement

This study utilized a much bigger data set than was used in the generation of the HCM 2010 or HCM 6 roundabout single-lane capacity models. For the HCM 2010 model, 318 data points from 18 roundabout approaches at 11 roundabout sites were used. For the HCM 6 model, 819 data points from 24 roundabout approaches at 10 roundabout sites were used. For the Louisiana locally calibrated model, 1,696 data points from 20 roundabout approaches from 9 roundabout sites were used.

It was observed that the HCM 6 capacity model overestimated the capacities at Louisiana sites, but the HCM 2010 capacity model generated much closer capacities for a given circulatory volume. The local calibrated model used an exponential regression model consistent with the HCM models, but with an intercept based on local driver behavior of follow-up times. Because the form and input parameters of the model are similar to that of the HCM models, it is anticipated that implementation should be straightforward. The recommended locally calibrated model for Louisiana single-lane roundabouts is as shown below (Equation [16] in the report):

$$Q = 1072.3e^{(-0.0009)v_{c,pce}}$$

where, Q = C_{pce}= Approach capacity in passenger car equivalent per hour, and
$V_{c,\,pce}$ = Circulatory volume in passenger car equivalent per hour.

Additionally, the study used SIDRA to tune the environment factor (EF) that would generate corresponding approach lane capacities closely matching the field-observed capacities, and other parameters like delay, LOS, and queue length. The result showed a mixed output with capacities and queue lengths close to the field derived estimates at EF of 1.06, while the remaining parameters delay, and LOS, showed estimates close to the field derived estimates at EF of 1.2. The average age of roundabouts used for the SIDRA analysis is eight years.

To quote directly from AKÇELIK [20]: “On the basis of the US HCM Edition 6 model, the Environment Factor values used are as follows:

- 1.05 for single-lane roundabouts (both approach road and circulating road have one lane),

- 1.05 for roundabouts with mixed single-lane and multilane approach and circulating road arrangements (either approach road or circulating road has one lane and the other road has two or more lanes), and
- 1.2 for multilane roundabouts (both approach road and circulating road have two or more lanes).*"*

This study determined Louisiana drivers have higher follow-up times and critical headways than the national averages: Louisiana (follow-up time of 3.36 seconds and critical headway of 4.76 seconds); HCM 2010 (follow-up time of 3.20 seconds and critical headway of 5.10 seconds); and HCM 6 (follow-up time of 2.60 seconds and critical headway of 4.70 seconds). Ultimately, this indicates that Louisiana is expected to have slightly lower capacity estimates than the national average, and perhaps is indicative of why an EF of 1.06 was obtained for single-lane roundabouts (with an average age of eight years) versus the EF of 1.05 suggested by AKÇELIK [20] for new builds. Lower EF values generate higher capacities, and higher EF values generate lower capacities. Generally, EF values tend to reduce over time.

For single-lane roundabouts, an EF of 1.0 is recommended for the Design Year, which is 20 years from the Build Year [20]. Considering an EF of 1.06 for an average 8 years, extrapolating to a Build Year of zero, results in an EF of 1.10.

Taking all the above into consideration, this study recommends a Build Year EF value of 1.1 if the HCM 6 model is used in SIDRA.

It will be further beneficial if a more innovative method of collecting field-observed delays, LOS, and queue lengths can be implemented and compared with SIDRA generated values to validate this EF recommended value of 1.1 for the Build Year. The recently purchased INRIX XD probe data may offer some benefits in exploring an innovative method of data collection.

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Introduction

Background

While indispensable to our roadway system, intersections are highly crash prone zones. They present complex traffic situations for planners, designers, and drivers alike. Intersections create opportunities for an interface between vehicles, vehicles and pedestrians, and vehicles with bicycles thus resulting in a myriad of scenarios with a high crash potential. In the United States (US), over the last several years, intersections have represented on average one-quarter of all traffic fatalities and roughly one-half of all traffic injuries [1]. Angle crashes account for over 40% of fatal crashes at intersections, left turn crashes account for over 20% of fatal crashes at intersections, and pedestrians/bicyclists crashes account for 25% of fatal crashes at signalized intersections [2].

Traffic flow at intersections has been mainly regulated by traffic control devices such as traffic signals and stop signs in the US, but safety concerns remain. In response to the grim safety problem, the Federal Highway Administration (FHWA) introduced "Innovative Intersections" to gradually mitigate the problem. The concept was to provide innovative means of reducing crashes or crash risks at intersections. These innovative intersections were categorized under three broad groups: circle-based, cross-over-based, and U-turn-based intersection innovations [3]. The circle-based innovations primarily constitute modern roundabouts. Restricted crossing U-turns (RCUTs) and median U-turns belong to the U-turn-based innovations and cross-over based improvements include grade-separated treatments that separate different directions of traffic.

Roundabouts are arguably the fastest growing intersection improvement type within the United States. Usage of modern roundabouts, as an alternative to traffic control devices, is becoming more popular in the US as a result of beneficial design modifications and safety improvements. Roundabouts at intersections have resulted in 35% reduction in all crashes and 76% reduction in injury crashes [4]. Severe crashes and fatal injuries are rare, but one study reported 89% reduction in these types of crashes [5], while another study reported 100% reduction in fatal crashes [6].

Problem Definition

The Highway Capacity Manual 2010 (“HCM 2010”) and the Highway Capacity Manual Edition 6 (“HCM 6”) [7] provide design guidelines for roundabouts. Specifically, features including the number of lanes, diameter of roundabout, estimated capacity, approach geometry, and multi-use paths are determined using these guidelines. Both editions of the Highway Capacity Manual (HCM) provide models to estimate the approach lane capacity of roundabouts, depending on the observed number of circulatory vehicles. Circulatory vehicles are the vehicles observed within the circular path around the central island of a roundabout. An approach lane is the lane that vehicles will exit from to enter the roundabout. The models provided by both HCM editions are based on nationally conducted studies to evaluate driver behavior at roundabouts within the United States. While both models fairly provide roundabout capacity estimates, the models are thought to be generic and not customized for specific geographic regions or locations, since driver behavior patterns are often uniform in specific regions and not across a country. Geometric features are decisions made by the designer and can be changed to suit the need of the roundabout location. Driver behavior, on the other hand, is intrinsic to the location and cannot be modified by the designer. The designer rather needs to make provision in the roundabout design to accommodate the observed driver behavior. Driver behavior comprises alertness, aggression, caution, driver response times, etc. Other factors that affect roundabout performance include visibility, interference by pedestrians, standing vehicles, parking, and a host of other factors that cannot be explicitly modeled. To reduce the magnitude of the unknowns, or those variables that cannot be directly catered for capacity models, the HCM recommends that agencies calibrate roundabout capacity models to their regions to capture and make provision for the nature of driving behaviors common amongst locals.

Additionally, SIDRA—a widely used software in US practice and approved by the FHWA for roundabout analysis [8]—uses an environmental factor as a general parameter to allow for the effects of factors such as driver aggressiveness and alertness (driver response times), the standard of intersection geometry, visibility, operating speeds, sizes of light and heavy vehicles, interference by pedestrians, standing vehicles, parking, buses stopping, and a host of other factors that have not been modeled explicitly [9]. The choice of value used as the environmental factor must closely mirror conditions at the roundabout site; otherwise, the roundabout may underperform, and queues and delays will be in excess. The state of Louisiana currently uses environmental factor values that have not been validated to existing conditions at roundabout sites. Owing to this, roundabouts developed with SIDRA and using the Louisiana Department of Transportation and Development's (LaDOTD) adopted environmental factor may or may not be performing at optimal levels, thus the need to determine roundabout capacities through field investigations and assess their performance.

In subsequent paragraphs, this report discusses circular intersections and delves more into roundabouts, types and features, and explains in detail the methods adopted by this research to calibrate the roundabout capacity models in the HCM, to better suit Louisiana conditions and driver behavior, to allow for optimal roundabout performance and decrease costs and inconvenience associated with travel delays. Assessment of the current convention used in SIDRA analysis by DOTD will also be validated and reported in this report.

Louisiana Roundabouts

Effective from August 2011, the DOTD sanctioned the adoption of roundabouts within the state by publishing an Engineering Directives and Standards Manual (EDSM) [6]. The policy set forth the justification, design, and approval methods for roundabouts and applies to roundabouts built as part of state highways or local roads.

Prior to approval for the installation of a roundabout, a justification and a roundabout report are to be developed. The justification aims to provide a sound engineering reason to support the decision to install the roundabout at the chosen site. It includes a cost benefit analysis and a capacity analysis comparison to prove financial viability and convenient user operability. The roundabout report is simply a detailed investigation and documents the present traffic and physical roadway features of the chosen site. This report often requires the approval of the state Traffic Engineering Division administrator.

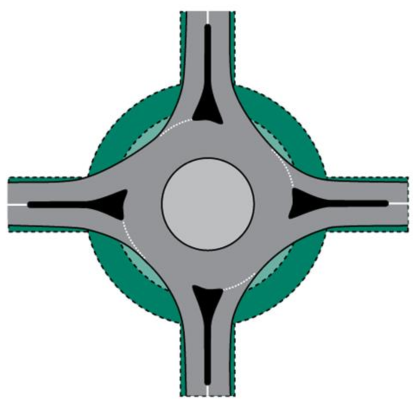
According to DOTD records as of February 2019, there were 49 state-built roundabouts, 3 local-built roundabouts, 105 state-proposed roundabouts, and 11 local-proposed roundabouts. These reflect plans and present situations for all nine DOTD districts within Louisiana [6]. A comprehensive list of state-built roundabouts in Louisiana, their status, and locations are added in Appendix A.

Literature Review

Roundabout Description

Circle-based intersection improvements include traffic circles or rotaries and roundabouts. These adopt a circular configuration to ease congestion and slow down vehicles moving through intersections thus improving traffic flow and safety significantly. Traffic circles differ from roundabouts in three main ways: size, allowable approach speeds, and approach path geometry. Modern roundabouts are often smaller than traffic circles. Figure 1 shows the difference in the size of a modern roundabout superimposed with a traffic circle. Also, modern roundabouts do not permit vehicle speeds exceeding 25 mph, but traffic circles allow approach speeds between 30 mph and 35 mph. Finally, vehicles travel a straight path to enter a traffic circle but navigate a gentle curve to enter roundabouts. The geometric curvature within the approach is a speed control measure. Figure 1 visually expresses this additional difference between roundabouts and traffic circles.

Figure 1. A superimposed image of a roundabout (in grey) and traffic circle (in green) [10]

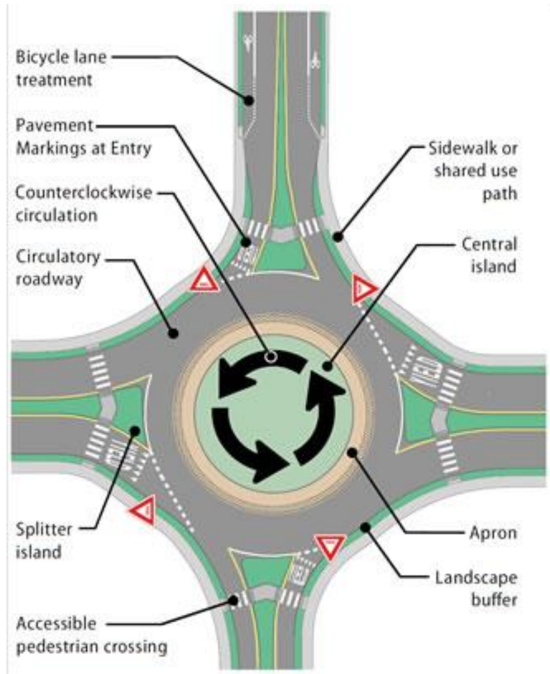


Features of Roundabout

A roundabout is an intersection traffic control feature with a circular layout designed to control the right-of-way of vehicles [3]. They have a central raised island around which vehicles navigate. These islands often constitute aprons to provide sufficient room to facilitate turning/curving movements of long vehicles around the central median.

Triangular splitter islands exist to slow and direct traffic [4]. Figure 2 shows a typical roundabout layout with a central island, apron, splitters, and a crosswalk.

Figure 2. A roundabout showing common geometric features [3]



The circular path within the roundabout is termed the circulatory lane(s) and the roads leading to the roundabout are the approach lane(s).

The first circular intersections installed within the US took the form of traffic circles. Over time, operational and safety concerns rendered this design detrimental. Circular intersection designs were no longer patronized by cities and states in the US until the late 90s when the concept of modern roundabouts penetrated the US transportation sector. There is currently an increasing preference for roundabouts over other traffic control devices by various state departments of transportation (DOTs). Owing to this, concerns regarding their efficiency have also increased. Several research—past and on-going—are being conducted to improve the design, construction, operations, and maintenance of roundabouts across the country.

Roundabouts are classified into three groups: mini-roundabouts, single-lane roundabouts and multilane roundabouts. Mini-roundabouts have islands that are entirely traversable or flushed with the road surface with internal diameters below 90 ft. These typically serve not more than 15,000 veh/day. Single-lane roundabouts operate a single-approach lane, have a raised central island with internal diameters between 90 ft. and 180 ft., and serve up to 25,000 veh/day. In the case of multilane roundabouts, there exists a raised central island, with internal diameters as high as 300 ft. They have two or more lanes on at least one approach and accommodate well over 25,000 veh/day. The allowable maximum approach speed of roundabouts increases with size: mini-roundabouts require the least approach, through, and exit speeds, while multilane roundabouts support the highest travel speeds, often not exceeding 30 mph [5].

Roundabout Concepts & Definitions

Relevant concepts or terms common to roundabout capacity discussions are briefly explained to aid in the understanding of the literature discussed in the report.

Gap Acceptance

Gap acceptance refers to the concept of a driver assessing a traffic stream for an acceptable gap to make a maneuver into the traffic stream. In the case of roundabouts, it refers to the decision for an approach vehicle to find an appropriate or acceptable gap to merge into the circulatory traffic [11].

Critical Headway

Critical gap or headway refers to the least duration or headway within the circulatory traffic that an approaching vehicle will consider sufficient to safely merge into or join the circulatory path.

Follow-Up Time

Follow-up time or headway describes the headway between successive vehicles in the approach lane that enters the roundabout using the same gap in circulatory flow.

Capacity Estimation

At present, the primary guide for the design of roundabouts is the Highway Capacity Manual (HCM). The capacity model in the HCM is based on results from the empirically observed performance of roundabouts in the US provided by the National Cooperative Highway Research Program (NCHRP) Report 572: “Roundabouts in the United States”

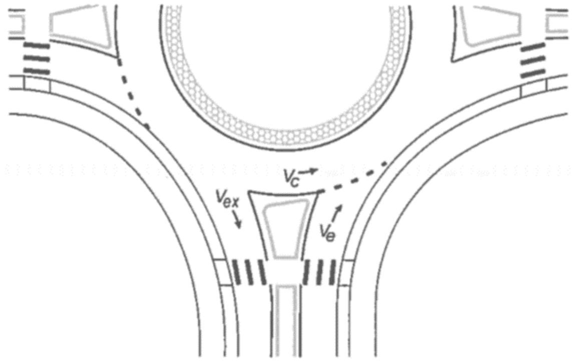
[12]. The HCM procedure derives its approach capacity from a statistical (regression) analysis and recommends a local calibration procedure using gap-acceptance parameters.

Gap acceptance parameters considered for capacity models are critical headways and follow-up times. Gap acceptance parameters are largely dependent on driver behavior and significantly vary regionally. This emphasizes the need to extract and evaluate data locally to calibrate the HCM's capacity models for the operational design of roundabouts.

Capacity models are often developed using statistical regression, deducing an inherent relationship between a known quantity and another. In the case of intersections, roundabouts inclusive, relationships between geometric features and capacity can be determined using a regression model. Analytical methods may be used in place of regression models. Analytical methods differ from regression models such that they reconcile traffic flow theories with field measures of driver behavior to predict capacity. When used independently, analytical methods are a better estimation of models as they incorporate observed behavior [12]. The HCM models however incorporate both lane-based analytical regression and gap acceptance models.

Three quantities directly impact the capacity of roundabouts: approach flow, circulating flow and exit flow. Approach flow represents the number of vehicles approaching the roundabout for a specific approach lane. Circulating flow refers to vehicles traversing the circular path within the roundabout. It is considered the primary circulating flow. Exit flow encompasses vehicles leaving the roundabout. Vehicles exiting the roundabout are considered a constituent of a secondary circulating flow: even though these vehicles do not directly interfere with the path of travel of approach vehicles, they influence an approach driver's decision to enter the roundabout. The term circulating flow will be used only in reference to circulating vehicles and not exit vehicles, which are secondary circulating traffic. An increase in any one of these flows, directly impacts the other two. Optimum roundabout operation implies a balance between these three quantities. Figure 3, obtained from the HCM 6, graphically demonstrates these three competing quantities.

Figure 3. Movements at roundabouts [12]



Analyses performed for roundabouts differ with respect to the number of lanes. Table 1 summarizes the two categories and their descriptive features (corresponding number of lanes).

Table 1. Classes of roundabouts and descriptive features

Number of Approach Lanes	Number of Circulatory Lanes	Roundabout Type
1	1	Single-Lane Roundabout
1	2	Multilane Roundabout
2	1	Multilane Roundabout
2	2	Multilane Roundabout

Studies Conducted within the US

NCHRP 572 (National Study)

The NCHRP Report 572 [11] is a summary of the NCHRP 3-65, "Applying Roundabouts in the United States," [13]. The increased interest in roundabouts across the US since the 1990s emphasized the need to advance scientific knowledge on roundabout design methods and performance improvement. This report was initiated to provide safety and operation tools local to US conditions for roundabout development and curtail the continued reliance on foreign/international roundabout development practices. As part of the study, national roundabout site inventories were updated, a database of data was created to support future research, safety prediction models for roundabouts were developed, and an updated HCM capacity model was developed. The sites used included locations in Colorado, Indiana, Virginia, Washington, New York, and Vermont. The

Maximum Likelihood Estimation (MLE) method was used to extract critical headways. The MLE assumes that a driver's largest rejected gap is consistently less than the accepted gap. The follow-up headways were observations of two consecutive approach vehicles utilizing the same gap in circulatory flow. An additional condition was for observations to have taken place during queueing conditions with a maximum move up time not exceeding six seconds. This was the basis for the development of the HCM 2010 capacity model.

FHWA-SA-15-070 Report (National Study)

The FHWA report [14] was primarily developed as an update to the earlier study used for the NCHRP 572 report. The study had fewer limitations and more sites to consider and served as the basis for the capacity model to be used in the HCM 6. The existing inventory of US roundabouts was updated and expanded upon, and a summary of geometric parameters and operational performance was provided. The capacity estimation and subsequent model development effort for this project focused on driver behavior parameters and geometric parameters. Similar to the NCHRP report 572, a maximum move up condition was imposed on the critical headways considered. However, the follow-up time estimation was different from what the NCHRP report utilized: the effect of exiting vehicles was excluded. The follow-up time gap for consecutive entering vehicles was included only when there were no potential perceived conflicts from vehicles exiting the circulatory path.

The geometric properties assessed were the inscribed circle diameter, average lane width, approach angle, and splitter island width. A correlation test was performed to draw the relationship between these physical features and the performance of the roundabout.

Florida

The state of Florida is one of the most recent to undertake local calibration of roundabout capacity models. Their work [15] focused on improving the HCM 6 models, using the gap acceptance parameters measured for Florida conditions. Similar to the national efforts, both single-lane and multilane sites were included in the work and queueing conditions were used. The study considered the influence of lane position in multilane roundabouts, a scenario that affects circulating gaps depending on the position of the approach vehicles in the left or right lane.

Wisconsin

Wisconsin considered two locations to perform capacity estimation of roundabouts [16]. For the gap parameters, they describe valid data points as those with queues that were at least five vehicles long throughout a full minute. The methods utilized were consistent with NCHRP 572 procedures, using the MLE method and considering only vehicles that rejected at least one circulatory gap before proceeding into the roundabout. Documentation included nine geometric properties of both sites considered.

California

California was one of the earliest states to calibrate HCM capacity models [17]. Ten sites, mostly made up of single-lane roundabouts, were evaluated. The MLE was used to extract critical gaps and follow-up times were obtained directly from recorded time events.

Oregon

The City of Bend’s Transportation Implementation Plan recommended that roundabouts be the prioritized or preferred option for intersection improvements [18]. This necessitated local roundabout capacity calibration to ensure the most efficient and safe designs of these intersections. Their single-lane models were calibrated to local conditions using extracted gap acceptance parameters, and the multilane models were consistent with the NCHRP 572’s findings, for which reason that model was adopted for use for multilane roundabout designs.

Georgia

Roundabouts are known to be common in Georgia. As part of the Georgia Department of Transportation’s efforts to improve roundabout designs, three roundabouts were identified and selected for the calibration efforts [19]. It is noteworthy that, one site had five legs instead of the usual four-leg/approach layout found at most sites used for similar studies. Data collected from these sites were reduced and analyzed to determine critical gaps and follow-up headways. Similar to other studies, the MLE method was used to compute the critical gaps, and the follow-up headways were directly computed from timestamps collected in the field. Weighted averages were determined for both measures of gap acceptance and the HCM calibration methodology was utilized.

The HCM 6 Model

Published in 2016, the HCM 6th edition [12] is a nationally accepted guide for transportation engineering that provides methods for measuring highway capacity. It specifies concepts, performance measures, and analysis techniques for assessing the operation of highways.

Chapter 22 of the HCM 6 exclusively describes roundabouts and covers capacity and level-of-service concepts, motorized vehicle core methodology, pedestrian and bicycle modes, and applications. An update to the NCHRP Project 03-65 was developed through efforts of an FHWA-sponsored project that gathered data from roundabouts across the United States—24 approaches at single-lane roundabouts and 37 approaches at multilane roundabouts were considered for this project. The NCHRP project, which was completed in 2016, aimed to develop methods to measure the safety and operational performance of roundabouts and refine design procedures used to develop roundabouts. The procedures provided by the HCM 6 for roundabout analysis are based on recommendations from the update to the NCHRP project. The HCM 6 proposes models for all four roundabout types listed in Table 1. The models however fail to cater for roundabouts with more than two lanes within the circulatory or approach paths. The models also do not sufficiently provide for roundabouts with high pedestrian or foot traffic. In the following paragraphs, the capacity models defined by the HCM 6 for both single-lane roundabouts and multilane roundabouts are thoroughly discussed.

Single-Lane Roundabouts

These operate as one approach lane, as well as one circulating lane. They may have three or four legs or approaches, but each will only constitute a single-lane. The HCM 6 suggests Equation [1] below to estimate the capacity at such roundabouts.

$$c_{e,pce} = 1{,}380e^{(-1.02\times10^{-3})v_{c,pce}} \quad [1]$$

where,
$C_{e,pce}$ = lane capacity, adjusted for heavy vehicles (pc/h), and
$V_{c,pce}$ = circulating flow rate (pc/h)

The capacity model in Equation [1] was determined using observations from the NCHRP project. The data obtained from the 24 single-lane approaches showed significant variation within and across regions. Therefore, the HCM cautions designers to locally calibrate this model to best reflect local driver behavior and distinguish from the generic national model. Measures of driver behavior focus on alertness and aggression and are obtained through the critical headway and follow-up headway [12].

Multilane Roundabouts

Multilane roundabouts present a more complex analysis method. This class of roundabouts has more than a single-lane in at least one approach, and two circulatory lanes, thus the complex nature of capacity analysis. A single approach lane within the approach flow is considered critical and used for analysis. The simple gap acceptance model does not sufficiently account for all the intricate scenarios that occur at multilane roundabouts. Different capacity models exist for all three classes of multilane roundabouts described in Table 1. Similar to single-lane roundabouts, local calibration is strongly advised for multilane roundabouts.

Two-Lane Approach Lanes with One Circulatory Lane

$$c_{e,pce} = 1{,}420e^{(-0.91\times10^{-3})v_{c,pce}} \qquad [2]$$

where, the variables are as previously defined in Equation [1].

One-Lane Approach Lanes with Two Circulatory Lane

For one lane approach lanes with two circulating lanes, Equation [3] is used to obtain approach capacities.

$$c_{e,pce} = 1{,}420e^{(-0.85\times10^{-3})v_{c,pce}} \qquad [3]$$

where, the variables are as previously defined in Equation [1].

Two-Lane Approach Lanes with Two Circulatory Lane

Field data suggest a notable difference between capacities for right lanes and left lanes—it is observed that left lane traffic often has a longer critical lane than right lane traffic, thus a lower capacity for the left lane [13]. For this reason, two different models are developed for capacity projections in roundabouts with two-lane approach lanes conflicted by two circulatory lanes. Equations [4] and [5] represent capacity models for right and left approach lanes, respectively.

$$c_{e,R,pce} = 1{,}420e^{(-0.85\times10^{-3})v_{c,pce}} \qquad [4]$$

$$c_{e,L,pce} = 1{,}350e^{(-0.92\times10^{-3})v_{c,pce}} \quad [5]$$

where,
$C_{e,R,pce}$ = right lane capacity, adjusted for heavy vehicles (pc/h),
$C_{e,L,pce}$ = left lane capacity, adjusted for heavy vehicles (pc/h), and
$V_{c,pce}$ = circulating flow rate (pc/h).

HCM 2010 Model

The HCM 2010 capacity model is a previous or older model relative to the HCM 6 model. This was developed based on the original NCHRP 567 report on roundabouts in the United States. Similar to the HCM 6, this model incorporates a non-linear empirical regression model and a gap acceptance (analytical) model for both single and multilane roundabouts [13].

Where roundabouts are located near traffic control devices including traffic control signals and crosswalks, the effects of these are not catered for by the capacity models.

Further, the model is unable to analyze roundabouts with more than two circulatory lanes—it is developed for isolated roundabouts with up to two approach lanes and one bypass lane for each approach [13].

Single-Lane Roundabouts

The HCM provides Equation [6] as the capacity model for single-lane roundabouts.

$$c_{e,pce} = 1{,}130e^{(-1.\ 0\times10^{-3})v_{c,pce}} \quad [6]$$

where,
$C_{e,pce}$ = lane capacity, adjusted for heavy vehicles (pc/h), and
$V_{c,pce}$ = circulating flow rate (pc/h).

Multilane Roundabouts

As described earlier, multilane roundabouts present significantly complex situations and varying layouts. To meet the specific needs of the different configurations of multilane roundabouts, three different models are provided by the HCM 2010.

Capacity for a Two-Lane Approach conflicted by a Single Circulating Lane

Equation [7] is used to determine the capacity for a roundabout with two approach lanes conflicted by a single circulatory lane.

$$c_{e,pce} = 1{,}130e^{(-1.0\times10^{-3})v_{c,pce}} \qquad [7]$$

where,
$C_{e,pce}$ = lane capacity, adjusted for heavy vehicles (pc/h), and
$V_{c,pce}$ = circulating flow rate (pc/h).

Capacity for a One-Lane Approach conflicted by Two Circulating Lanes

Equation [8] provides the capacity of a one lane approach conflicted by two circulating lanes.

$$c_{e,pce} = 1{,}130e^{(-0.70\times10^{-3})v_{c,pce}} \qquad [8]$$

where, all variables are as previously defined.

Capacity for a Two-Lane Approach conflicted by Two Circulating Lanes

Where two approach lanes are conflicted by two circulating lanes, two capacity values are obtained to represent each of the two approach lanes—the right approach lane and left approach lane.

Equations [9] and [10] represent the right lane capacity and left lane capacity, respectively

$$c_{e,R,pce} = 1{,}130e^{(-0.70\times10^{-3})v_{c,pce}} \qquad [9]$$

$$c_{e,L,pce} = 1{,}130e^{(-0.75\times10^{-3})v_{c,pce}} \qquad [10]$$

where,
$C_{e,R,pce}$ = right lane capacity, adjusted for heavy vehicles (pc/h),
$C_{e,L,pce}$ = left lane capacity, adjusted for heavy vehicles (pc/h), and
$V_{c,pce}$ = circulating flow rate (pc/h).

Alternative Tools for Roundabout Capacity Modelling

Other methods of capacity modeling are suggested by the HCM at the discretion of the engineer. Alternative modeling tools include all other capacity deduction methods other than the models provided by the HCMs. Common alternative tools for capacity modeling are developed either as deterministic intersection models or stochastic network models.

Deterministic intersection models consider flow and geometric characteristics of a roundabout to compute capacities. Specifically, number of lanes, lane width, inscribed circle diameter, etc. are used to derive the capacity of the roundabout. The shortfall of this is, the interaction effects of a roundabout and other (nearby) intersections are ignored, thus limiting the application of deterministic models.

Stochastic network models assess individual vehicles in the traffic stream. Simulations are performed for each vehicle, its car-following, and lane-choice and gap-acceptance decision. These models are preferred over deterministic models because they are capable of modeling entire network of intersections, while considering interactions between these. Given the broad nature of their output or results, their data requirements are significantly higher, and challenging to meet. These serve as the basis of microsimulation tools.

SIDRA Roundabout Capacity Model

SIDRA INTERSECTION (SIDRA) is the most popular roundabout capacity design software within the US. It is used for planning and design purposes by several DOTs and referenced in NCHRP reports, the HCM, and several TRB publications [20].

The software provides options for analyses to be performed either based on the HCM 6 and 10 models or the Australian roundabout capacity model (the SIDRA Standard Model). Even though the HCM provides for base conditions for capacity assessment, it refers users to alternative tools for more complex conditions beyond its scope. The following conditions are specified by the HCM as beyond its scope:

i. Adjacent signals or roundabouts,
ii. Priority reversal under extremely high flows,
iii. High pedestrian or bicycle activity lanes on an approach, or
iv. Flared approach lanes [12].

An alternative tool like SIDRA caters to these conditions that are not adequately included in the scope of the HCM, thus the reliance on SIDRA for more detailed roundabout capacity analyses. Such detailed analysis using SIDRA provides output including the various relevant measures [20].

Firstly, SIDRA is developed to analyze road networks with up to 20 sites that may include roundabout corridors, signalized intersections and roundabouts, and sign controls. The results from such analyses provide estimates of capacity, expected delays, queue lengths, fuel consumption, and other relevant performance measures. These results may be lane specific, movement phase dependent, or related to movement groupings, i.e., vehicles or pedestrian traffic.

Further, a Design Life analysis may also be conducted in SIDRA to predict traffic growth and assess its impact on the performance of roundabouts. This falls under the demand and sensitivity considerations of the roundabout design. A Design Life and Build Year value are both chosen to enhance analysis. In the case of the Louisiana DOTD, 3 years from the traffic study is selected for Build Year, and 20 years for Design Year. The design year is measured from the Build Year. A cumulative demand projection of 23 years is therefore used [21].

SIDRA allows for “in-program” calibration of its models to serve varying local conditions. It provides two parameters to support calibration: the environment factor and approach/circulating flow adjustment. Modification to these parameters, directly impact gap-acceptance parameters and consequently affects the capacity of roundabouts [22].

The environment factor values commonly used are in the range of 1.0 to 1.2. These dictate how restricted the capacity model is. Restriction correlates with capacity: a higher environmental factor, which implies an increase in restriction, results in a lower capacity. As implied by the name, the environment factor is a measure of conditions within the environment of the roundabout that impacts the flow of traffic through the intersection. These conditions reflect typical driver response time and aggression, visibility, grade, speed, percentage of usage of roundabout by pedestrians and heavy vehicles, proximity, or existence of parking at the roundabout, etc. These conditions either favor the capacity estimates or reduce performance. While good visibility, low pedestrian traffic, and short driver response times positively affect capacity, the opposite reduce the throughput of roundabouts [22].

A value of 1.0 is known to mirror ideal or optimum flow conditions at roundabouts: drivers are familiar with the layout and navigate through the roundabout without difficulty, visibility is good, little to no pedestrian interface exists, few heavy vehicles use the roundabouts, etc. Australian roundabout research, based upon which the SIDRA program was first developed, suggests a significantly higher performance of Australian roundabouts relative to US conditions. Australia, therefore, assumes an environmental factor of 1.0. By default, SIDRA uses this value since it was first developed for Australia. This 1.0 Environmental Factor model is termed the SIDRA Standard Model. UK traffic characteristics are analogous to Australian conditions. Norwegian roundabouts are known to reflect 1.1 factor conditions [23]. Poland presents subpar capacities for multilane roundabouts with an environmental factor of 1.39, and 1.05 for single-lane roundabouts.

In the case of the United States, a lack of familiarity with roundabouts, decreased driver aggression, etc. account for lower capacities at roundabouts. Comparing US capacity values obtained from the HCM 2010 with output from SIDRA, it is observed that an environmental factor of 1.2 in SIDRA produces results similar to US records for both single-lane and multilane roundabouts. For the HCM 6, however, results are similar for SIDRA analysis only when 1.05 is used as a factor for single-lane roundabouts, and 1.2 for multilane roundabouts [20].

DOTD uses SIDRA INTERSECTION 6.1 for its roundabout analysis and uses an Environment Factor of 1.2 for Build Year Design and 1.1 for Design Year. Generally, the Build Year is approximately 3 years from the traffic study year and the Design Year is 20 years from the Build Year. The lower the Environment Factor, the more familiar the drivers are with the roundabout operations, and the higher the roundabout capacity.

Other Roundabout Capacity Estimation Tools

Manually estimating roundabout capacities can be tedious and time-consuming. Several commercial software/tools are available for engineers to use to expedite roundabout traffic analysis and capacity estimation [20].

Rodel is a commonly used roundabout capacity software that predicts operational performance for differing geometric conditions. The operating model was developed from several observations from 86 sites using about 11,000 minutes of maximum capacity operation. At least 10 DOTs and several cities in the US rely on this model to develop roundabouts [24].

Arcady is another roundabout design tool that links roundabout geometry to driver behavior to estimate capacities, queues, and delays. It offers a lane simulation feature that allows individual lane performance to be measured [25].

The Highway Capacity Software developed by the McTrans Center supports analysis of roundabouts with a limited range of volumes. It is entirely based on methodologies and equations recommended by the Highway Capacity Manual. It offers an optional gap acceptance parameter setting to facilitate both conservative and liberal estimation methods [26].

With a varying number of roundabout configurations, the German developed software, Kreisel offers many user-specific alternatives to utilize different roundabout capacity estimation procedures and methodologies. Kreisel also offers a means to compare results from different procedures [26].

Comparison of the HCM Models and SIDRA Standard Model

The HCM 2010 and HCM 6 models are very similar. The latter is an update to the HCM 2010, following update on NCHRP Project 03-65 Report. Both manuals provide roundabout capacity models developed from data collected for the NCHRP report. In both cases, the models are empirical (exponential regression) and incorporate a gap-acceptance feature. Both models take the general form:

$$Q = Xe^{(Y)v_{c,pce}} \qquad [11]$$

where,
Q = Approach lane capacity, adjusted for heavy vehicles (pc/h),
X = 3600/t_f,
Y = (t_c-0.5t_f)/3600, and
$V_{c,pce}$ = circulating flow rate (pc/h).

While the form of the models remains the same, the gap acceptance parameters, X and Y are different for the HCM 2010 and HCM 6 equations. X and Y are obtained from estimated parameters of follow-up times (t_f) and critical headways (t_c). Table 2 portrays the difference in the X and Y parameters for the HCM 2010 and HCM 6 models. The X parameter is directly proportional to the capacity, Q—a higher X value results in a higher estimate of capacity. For both single-lane and multilane models, the HCM consistently has a higher X parameter. It suggests that, for a fixed Y parameter, the HCM 6 predicts a larger capacity. The Y parameter is used as an exponential term, making it inversely proportional to Q. The impact of Y on the capacity is more pronounced in the HCM 2010 model since its Y is often lower. Table 2 shows the comparison of gap acceptance parameters for both models.

Table 2. Comparison of gap acceptance parameters, X and Y, for HCM models

Model	HCM 2010	HCM 6
X	1,130 (for both single-lane and multilane)	1,380 & 1,420 (single-lane and multilane)
Y	0.00075-0.001 (depending on roundabout type)	0.00085-0.00102 (depending on roundabout type)

The HCM models and SIDRA models are developed as regression models founded on the gap acceptance theory. Both are lane-based models that determine the capacity and performance of individual approach lanes. However, while the SIDRA model considers the influence of a roundabout's geometry, the HCM models do not. This offers a more "aggressive" analysis while using the Sidra Model.

Even though both models provide a medium for lane-based analyses, SIDRA considers the approach and circulating lanes in more detail than the HCM models do. Circulating lane flow rates are modeled in SIDRA to allow for unbalanced flows in the circulatory path. HCM uses a unit value to represent the total circulating flow rate without regard for variations in the individual lanes making up the circulatory path. This difference is eliminated for single-lane roundabouts since they have a single circulatory lane. SIDRA also includes a proportion of exiting flow to be added as the circulating flow, but the HCM models do not. However, both the HCM and SIDRA identify dominant and subdominant approach lanes for analysis [23].

To determine approach lane utilization, SIDRA uses lane utilization ratios to make up for unequal lane use. HCM achieves a similar computation using lane volume percentages. For degrees of saturation, both models use a critical lane v/c ratio for a multilane approach.

Also, both the HCM and SIDRA models provide model calibration procedures. The latter utilizes known follow-up times and critical headways values to do this. SIDRA provides approach-level calibration and intersection level calibration using an Environmental Factor value. The factor caters for conditions that may not be considered in modeling like driver aggressiveness and alertness, standard of intersection geometry, vehicle size, pedestrians, visibility, and other conditions that may influence the throughput of the roundabout [23].

In capacity modeling, some roundabout geometry parameters may be utilized. While SIDRA exhausts all such geometric features for its analyses, the HCM barely does. HCM only considers the number of approach lanes, approach lane disciplines and bypass lanes, as well as number of circulating lanes. SIDRA uses these features and more: average approach lane width, inscribed diameter, approach radius, approach angle and number of exit lanes are all considered [23].

Heavy vehicles significantly affect roundabout capacities. Where a roundabout is likely to be used by a large volume of heavy vehicles, capacities may be lower. The HCM models decrease capacity directly using a factor, to account for heavy vehicle usage. In the case of SIDRA, the circulatory flow rate is increased in isolation, to account for heavy vehicle usage within the circulating stream. The follow-up times and critical headway values are increased for heavy vehicles in the approach lane(s) [23].

Congestion is directly related to fuel consumption and emission values. Slower moving traffic tends to increase fuel costs and pollution. As capacity influences congestion, it may be of relevance to consider how the estimated capacity of a roundabout would affect user fuel efficiency, environmental pollution, and user operating costs. Whereas the HCM ignores this concept, SIDRA achieves this through a detailed vehicle power-based model using drive cycle data for each lane. Light and heavy vehicles, for reasons of variation in fuel economy, are separately analyzed.

Control Delay and Level of Service

Delays and consequent queues are primary performance measures of the level of service at intersections. Roundabouts, considered analogous to two-way and four-way stop controlled intersections, are no exception to this concept. Control delay describes delays caused by control devices like traffic signals and stop signs. The presence of an intersection control is thought to alter the duration of travel along a specific path. The difference in the duration with and without the intersection control is a measure of control delay. Control delay determines the level of service of a facility, in this case, a roundabout. Level of Service (LOS) is a qualitative measure of traffic flow rate at intersections. A scale that ranges from A to F is adopted to qualify the quality of traffic flow or conditions. A LOS of A represents ideal conditions where there is the maximum free flow of traffic, individual vehicle speeds are not affected by other vehicles in the stream, and drivers are often at liberty to move at desired speeds within the speed limits. LOS F represents worst case conditions where traffic flow is poor, and delay is the highest. A volume to capacity ratio is computed to measure LOS. Where the traffic volume exceeds the capacity, (v/c>=1.0), LOS is F. Control delay is measured and averaged in seconds per vehicle (s/veh). Table 3, obtained from the HCM, exhibits the accepted LOS criteria at roundabouts.

Table 3. LOS criteria for unsignalized intersections (including roundabouts) [12]

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

Objective

Primarily, this project sought to use local data to determine Louisiana's roundabout capacity and compare to software (SIDRA) outcomes that are currently used in the planning and design of modern roundabouts in Louisiana.

Specifically, the main objectives are:

i. Conduct a literature review on the roundabout capacity models as presented in the HCM 2010 and HCM 6, highlighting differences and similarities and comparing to SIDRA capacity estimation methods.

ii. Select candidate sites for local data collection to be used for parameter estimation.

iii. Compare parameters obtained from site observations to HCM 2010, HCM 6, and SIDRA outputs.

iv. Validate Environmental factor value used in SIDRA INTERSECTIONS for Louisiana purposes.

v. Make a recommendation on best practices to be followed in determining parameters that best reflect local driver behavior.

Scope

To meet the outlined objectives of this research, a preliminary assessment was conducted for several state-maintained roundabouts that had been constructed at the time of consideration and met the criteria for site selection. Consideration was given to both urban and rural locations to identify a possible difference in driver behavior for either location. Sites were also clustered into functional classes as well as DOTD districts to test for a possible difference in driving patterns across clusters. It is important to note that, the study was limited to only built roundabouts since the study's objective was to validate the existing methodology of capacity estimation and the utilization of SIDRA for roundabouts design. The selection criteria are further explained in this report. Only single-lane roundabouts were explored.

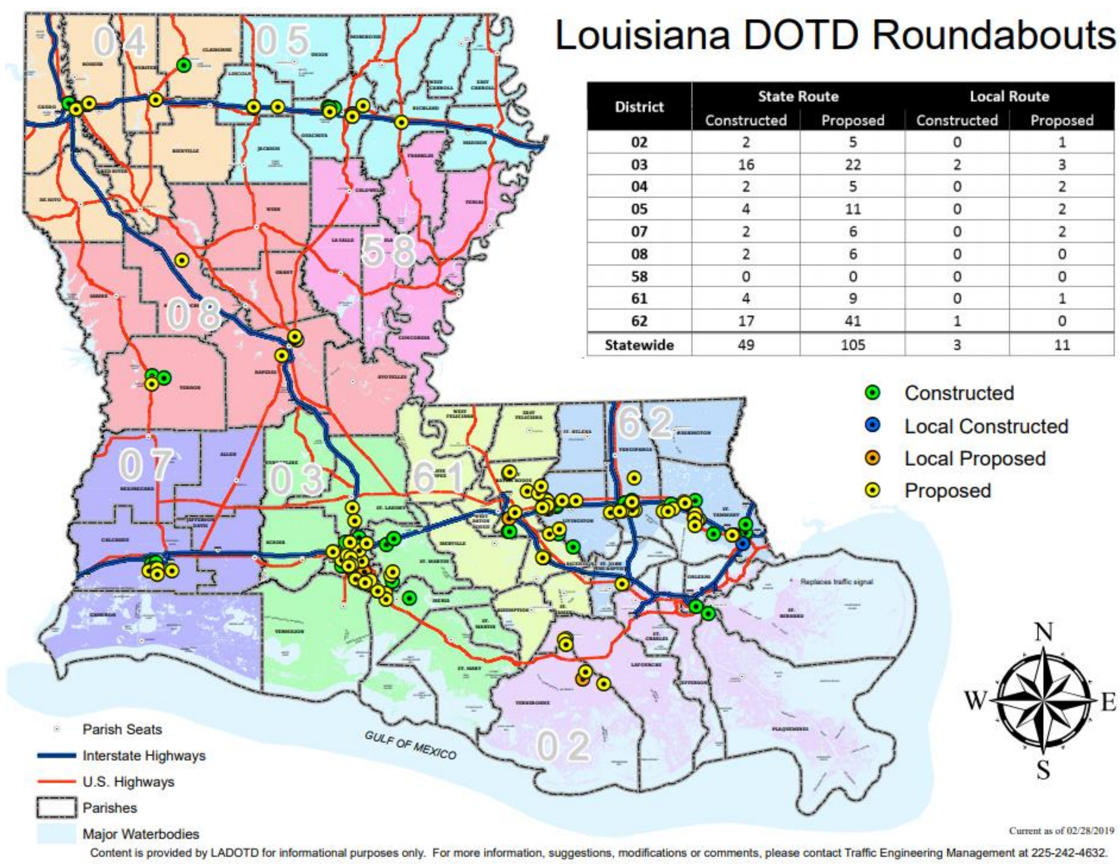
Methodology

This chapter presents a detailed description of the data used for this study and proceeds to explain the methodologies and procedures employed to obtain results for the report.

Roundabout Selection and Descriptions

Per the Louisiana DOTD's records [27], as of February 2019, there were 52 constructed roundabouts within the state: 49 along state routes and 3 along local routes. Figure 4 shows the location and types of roundabouts across the state.

Figure 4. Location and types of roundabouts within Louisiana



District	State Route		Local Route	
	Constructed	Proposed	Constructed	Proposed
02	2	5	0	1
03	16	22	2	3
04	2	5	0	2
05	4	11	0	2
07	2	6	0	2
08	2	6	0	0
58	0	0	0	0
61	4	9	0	1
62	17	41	1	0
Statewide	49	105	3	11

However, only the roundabouts meeting the following criteria were used for this study.

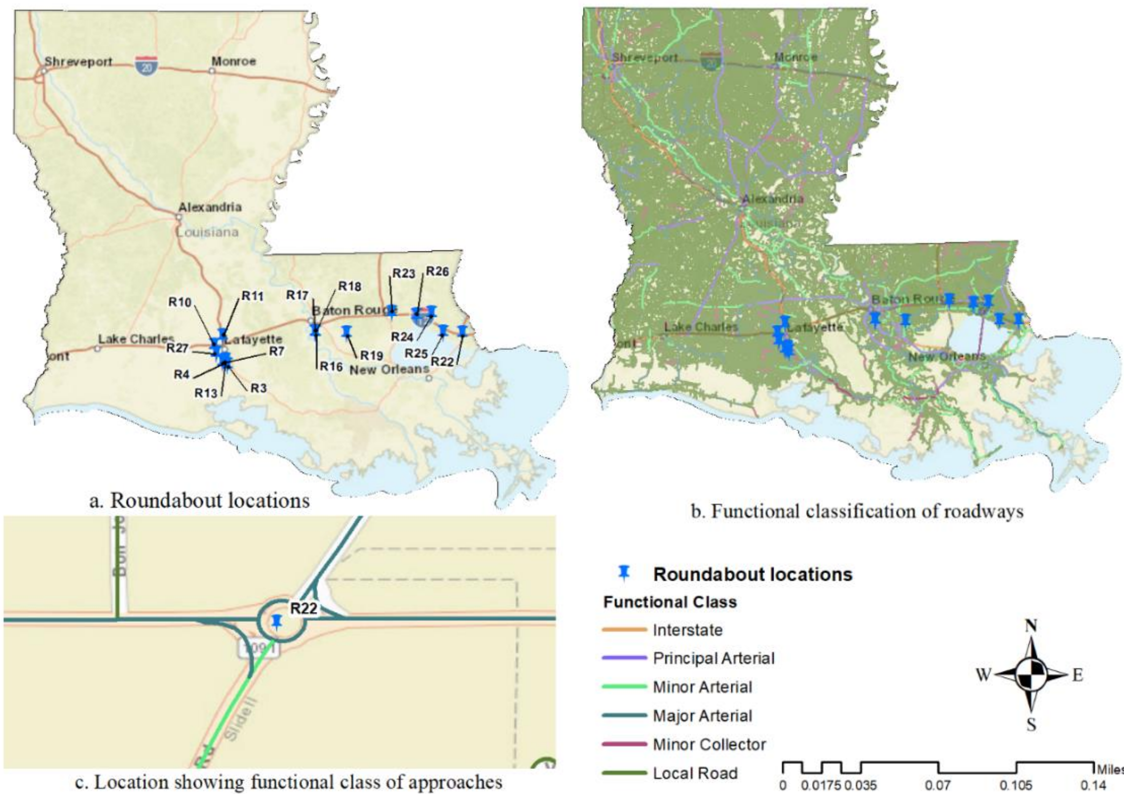
i. Single-lane for all approaches
ii. Nearly mutually perpendicular approaches
iii. Approximately flat longitudinal gradients on all approaches
iv. Negligible exposure or interference from pedestrians or cyclists
v. Experience congestion or queues for some portion of the day, preferably at least 15 minutes, on at least one approach
vi. Available as-built plans

Following the above criteria, 41 approaches from 17 roundabouts were selected for data collection for the study. It is important to note that, each approach is considered independently, therefore a selected roundabout could have between one to four eligible approaches. For roundabouts that did not have all four approaches included in the study, the lack of sufficient queuing during peak times was the reason for eliminating some of the approaches. Using Google Maps, a visual inspection of typical traffic conditions at each site, was used to validate queueing/congested conditions for at least 15 minutes, in accordance with the fifth criterion. Figure 5 shows an example of a selected roundabout in the Lafayette area.

Figure 5. Sample site (Source: Google Earth, accessed 04/12/2020)

While efforts were made to consider both rural and urban sites, only a single site meeting the criteria was located in a rural enclave—all others were in urban locations. Using LaDOTD's ArcGIS/Esri Functional System Maps, shown in Figure 6, each site's functional class was obtained and considered in the analyses [28]. Appendix B presents the chosen sites, age of roundabouts, all the approaches and their respective description with functional class.

Figure 6. Roundabout locations and ArcGIS functional classification map



a. Roundabout locations

b. Functional classification of roadways

c. Location showing functional class of approaches

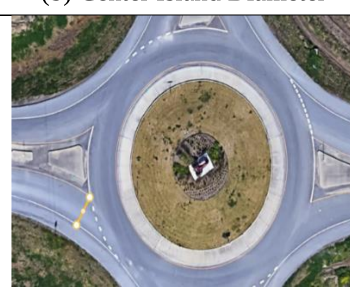
Data Collection

For each approach, the respective District Traffic Operations Engineer (DTOE) provided typical weekday peak times to be considered for data collection. These were further confirmed using the typical traffic feature of Google Maps. Cameras were installed (similar to the setup in Figure 7) at each site for a minimum of 48 hours in order to capture peak traffic flows for two mornings and two afternoons. Each camera was positioned within the island to directly face the approach of interest, capture all approaching vehicles, and capture circulatory vehicles as they conflict the approach path. For most approaches, identified AM peak flows occurred between 7AM and 9AM and PM peaks between 4PM and 6PM. Data were collected on days that had no road closures or work zones close to the roundabout, nor during any weather events that could impact the behavior of drivers. The videos obtained were used to determine actual observed approach volumes with corresponding circulating flow volumes, number of accepted gaps and number of rejected gaps for each qualified approach event, follow-up times, and percentage of heavy vehicles.

Figure 7. Camera installation setup

Using Google Earth and AutoCAD, the geometric features of each roundabout were measured. Geometric features assessed include inscribed circle diameter (ICD), Center Island Diameter (CID), Splitter Island Width, and Approach Lane Width. Figure 8 shows these measurements.

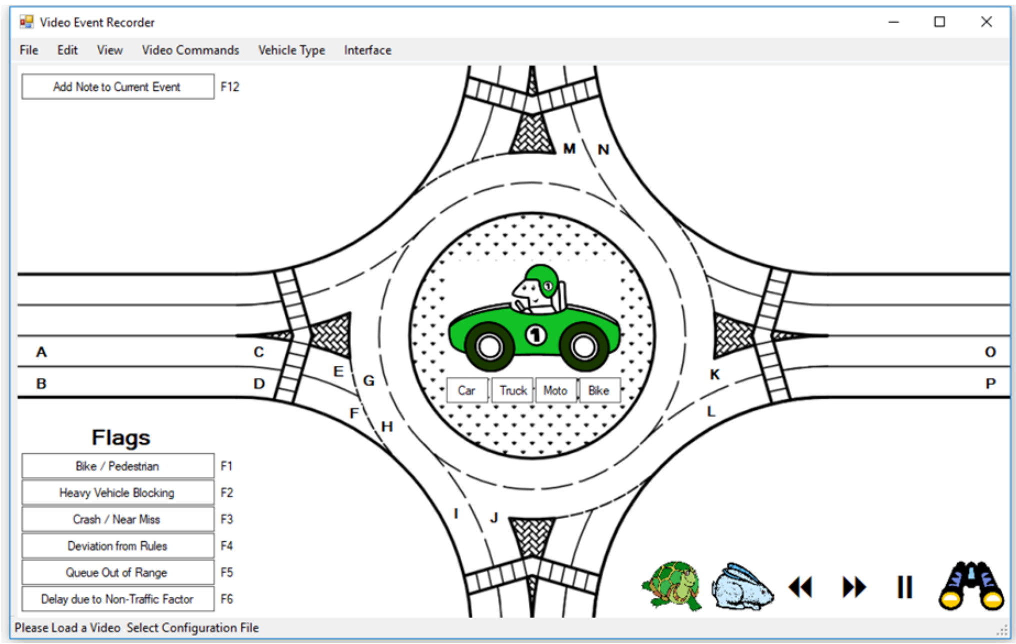
Figure 8. Geometric measures of roundabout

	
(a) Inscribed Circle Diameter (ICD)	(b) Center Island Diameter
	
(c) Splitter Island Width	(d) Approach Lane Width

Data Reduction

Data reduction was achieved using a roundabout event recorder (RER) developed by the University of Wisconsin's Traffic Operations and Safety (TOPS) Laboratory. Figure 9 shows the interface of the RER.

Figure 9. User interface of roundabout event recorder



The collected videos were reduced to one-minute bins defined by either of two criteria: a minimum queue or a maximum move-up time. The minimum queue criterion is met by observing at least two vehicles in a queue before the yield line for a full minute. The maximum move-up time criterion is satisfied when the interval between each vehicle departing from and the next vehicle arriving at the yield line does not exceed six seconds for the full minute under observation. The criteria for choosing one-minute bins ensured that only queueing conditions were selected. Using the roundabout event recorder, timestamps were extracted for all events of interest from the videos and a comma separated values (CSV) text file was obtained as the output from the RER. The RER allowed manual keystroking corresponding to the events of interest. Table 4 summarizes the keystrokes and the corresponding action or event for which it is used. The output file was manipulated using mostly Excel and R functions to estimate the critical headways and follow-up times from each event.

Table 4. Keystrokes used in data reduction

Key	Event
F	Approach vehicle arrives at yield line
H	Approach vehicle departs yield line
Q	Circulating vehicle as it conflicts the approach path
T	Circulating vehicle as it exits the circulatory path
1	Observed Vehicle = Car
2	Observed Vehicle = Truck or other long vehicle

Follow-Up Headway Estimation

The time difference between the departure times of vehicles successively entering the roundabout, results in the follow-up headway on condition that both vehicles accept the same gap in circulatory flow. For example, in Table 5, for an identified queue occurring between the passing of two circulating vehicles that crossed the approach lane at 7:30:07 and 7:31:06 consecutively, Event 1 is disqualified because it does not occur within the identified gap in circulatory flow. Event 2 occurs within the observed gap but does not produce any follow-up time since it is the first observed approach vehicle. Events 3, 4, and 5 produce follow-up times for consideration because they all accepted the same identified gap.

Table 5. Sample data for follow-up time estimation

Approach Vehicle/Event	Arrival Time (Keystroke F)	Departure Time (Keystroke H)	Follow-up Time
1	7:30:02	7:30:04	N/A
2	7:30:08	7:30:11	-
3	7:30:12	7:30:18	1 sec (F3-H2)
4	7:30:21	7:30:22	3 sec (F4-F3)
5	7:30:24	7:30:26	2 sec (F5-F4)

Critical Headway Estimation

The critical headway (t_c) is the least headway within the major traffic stream that a single approach vehicle can safely use as an entry or approach [12]. This implies that any headway less than the critical headway will be rejected by an approach vehicle, and all accepted headways will be larger than the critical headway. This theory only holds upon the assumption that all drivers behave consistently and rationally [11].

Similar to the NCHRP 572 and FHWA-Sa-15-070, the recommended method of evaluating critical headway is the Maximum Likelihood Estimation (MLE). Other methods, including the Troutbeck's or Raff's method, are well-known procedures of estimating critical headways at non-signalized intersections, but these methods are only as accurate as the level of consistency in driver behavior at the intersection [29]. The MLE determines parameter values such that the likelihood that an event estimated by the model results in data that is observed [30].

Using the extracted timestamps, for each approach vehicle, the gaps in circulatory flow accepted or rejected are computed for each site. For an event to qualify as a critical headway, the approach vehicle must first reject at least one gap before accepting any subsequent one. For single-lane roundabouts, the NCHRP 572 tested three methodologies of estimating critical headways: the first includes all observations of gap acceptance, the second includes only observations where an observed approach vehicle rejects at least one gap, and the third considers the second case scenario for situations where queuing was observed for an entire minute [11]. The adopted methodology for this study was similar to the second methodology, where the only imposed condition was the rejection of at least one circulatory gap before another gap was accepted.

Some observations included situations where a driver's largest rejected gap was greater than the accepted gap. Such events were eliminated from the analysis as the drivers were considered inattentive; otherwise, it would be impractical for a driver to accept a gap less than a previously rejected gap. Most studies also excluded such data from their analysis, for the same reason. However, a few studies have included such data in their analysis and used the largest rejected gap just less than the accepted gap instead of the true largest rejected gap which was larger than the accepted gap [31]. Further, there exists a theory that, with an increasing number of rejected gaps, drivers turn to be more aggressive and may finally accept a gap that they have rejected before [32]. There was sufficient data to move forward with the study despite excluding these scenarios which had the potential to skew the analysis.

Estimation of Capacity

For capacity analysis, the data from each site was reduced to obtain volumes for approach and circulating flows. Data bins were created, conditioned on a maximum move-up time of six seconds and a minimum observed queue for at least a full minute. Data binning was essential to reduce and extract small time units for queueing conditions, and subsequently, obtain associated approach and circulating volumes for each bin. The regression models to be developed required such data for plotting scatter plots and developing (regression) trends. Using Excel, a macro was developed to extract from the time-stamped data, all approach events where a vehicle followed within six seconds for at least a full minute. An excel function (count if) was subsequently used to count the number of approach vehicles and circulatory vehicles associated with the identified bin. While no bins were less than a minute long, a few exceeded one minute by a few seconds, to cater for vehicles that approached the yield line during the minute under consideration but exited a few seconds after the full minute had elapsed. Table 6 provides an example of the computations for the binning exercise.

Table 6. Data binning for vehicles arriving from 15:32:04 to 15:33:09

Vehicle/Event (n)		Arrival Time (F-Time Stamp)		Departure Time (H Time Stamp)
1	F	15:32:04	H	15:32:05
2	F	15:32:07	H	15:32:21
3	F	15:32:23	H	15:32:31
4	F	15:32:34	H	15:32:41
5	F	15:32:44	H	15:32:45
6	F	15:32:47	H	15:32:47
7	F	15:32:50	H	15:32:56
8	F	15:32:58	H	15:32:59
9	F	15:33:00	H	15:33:00
10	F	15:33:03	H	15:33:06
11	F	15:33:07	H	15:33:08
12	F	15:33:09	H	15:33:09
13	F	15:33:15	H	15:33:16

In the example above (from R8 East site), the 12 vehicles will make up the approach vehicles for that bin, and a count of timestamps from the circulatory data that fall within the same bin (15:32:04 to 15:33:09) are also extracted and counted as the circulating volumes for that bin. Vehicle 13 did not qualify as including it, which would cause the bin to further exceed one minute.

A count of vehicles for the approach and circulatory paths for an observed bin was converted to flow rates, following conventional practices in traffic engineering. Even though an observed bin may only be a minute long or slightly more, the equivalent hourly flow in passenger car units is required for analysis. The passenger car adjustment is achieved by applying a factor of 2.0 to all observed large vehicles (trucks, buses, etc.). To obtain hourly flows, Equation [12] is used, and sample results are shown in Table 7 with flow rates reported in the nearest whole numbers.

$$Hourly\ flow\ (\frac{pcu}{hr}) = \mathrm{a}\ \times \frac{60}{b} \qquad [12]$$

where,

a = number of vehicles observed (Va or Vc) in passenger car units (pcu), and

b = length of bin in minutes.

Table 7. Sample results from data binning

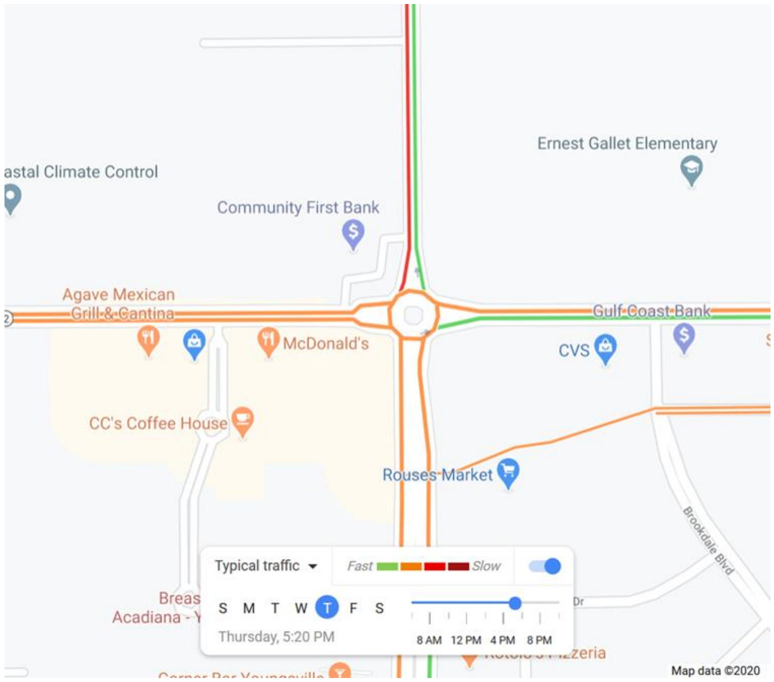
Bin No.	Length of Bin (mins)	V_a(pcu)	V_c(pcu)	V_a (pcu/hr)	V_c (pcu/hr)
1	1.10	18	7	982	382
2	1.06	14	6	792	340
3	1.04	12	4	692	231
4	1.00	10	3	600	180

Estimation of Delays, Level of Service, and Queue Length

The peak flows are worse case scenarios that are considered to measure system performance. Peak times were determined for each site using the HCM's recommended methodology, which focuses on estimating the highest volumes for the 15-minute periods within the peak hours. Such peak periods were further validated using Google Maps. Google Maps maintain a historical log of traffic patterns using a color themed layer activated at maps.google.com. Using the color code, it is easy to identify the most congested time of day at a location. This feature was employed to validate the theoretical/ field-observed and peak times. Figure 10 shows a snapshot of traffic patterns from Google Maps at the roundabout approach R4.

For most intersections, signalized or non-signalized, delays are a principal performance measure that influences the evaluation of the level of service of the intersection [33]. To estimate the traffic operational measures of effectiveness of the study sites, a control delay analysis was performed for each approach. This was done using field observations.

Figure 10. Snapshot from Google Maps showing typical traffic at 5:20PM on Thursdays at site R4



Theoretical Estimation of Delays

The HCM's Equation [13] was used for these computations. Equation [13] is bi-functional in traffic analyses such that it is also used to estimate delays for STOP-controlled intersections except for the concluding "+ 5" term, a modification to account for the YIELD control at roundabouts, that does not require drivers to completely stop at the approach line [12].

$$d = \frac{3600}{C} + 900T\left[x - 1 + \sqrt{(x-1)^2 + \frac{\left(3600/_{c}\right)x}{450T}}\right] + 5 \times \min[x, 1] \qquad [13]$$

where,

d = Average control delay (s/veh),

x = volume-to-capacity ratio of the subject lane,

c = capacity of the subject lane (veh/h), and

T = time period (h) (T = 0.25 for 15-min analysis).

The computations above are applicable per observed approach/site. To assess the intersection as a whole, the control delay is a weighted average of the delay per approach, weighted by the volume on each approach, as shown in Equation [14]. Only two of the roundabouts used in this study had all four approaches considered; therefore, only those two had a control delay, d, computed for the entire intersection.

$$d_{intersection} = \frac{\Sigma d_i v_i}{\Sigma d_i} \qquad [14]$$

where,
$d_{intersection}$ = control delay for the entire intersection (s/veh),
d_i= control delay for approach I (s/veh), and
v_i= flow rate for approach I (veh/h).

Level of Service

Given the computed control delay, the associated LOS is determined from Table 8 based on the volume-to-capacity ratio.

Table 8. Control delay and LOS

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

Queue Length

The 95th percentile queue length was used to estimate queue length at each approach in terms of number of vehicles. The queue length for each approach was estimated using Equation [15].

$$Q,95 = 900T\left[x - 1 + \sqrt{(x-1)^2 + \frac{(3600/_c)x}{150T}}\right] * \left(\frac{c}{3600}\right) \qquad [15]$$

where,
$Q,95$ = 95th percentile queue, veh,
X = volume-to-capacity ratio of the subject lane,

c = capacity of the subject lane (veh/h), and
T = time period (h) (T = 0.25 for 15-min analysis).

SIDRA Analysis

A consultant [34] was hired to develop the environmental factor (EF) required for roundabout analysis in Louisiana. The task involved the development of SIDRA models of existing roundabouts in Louisiana that were analyzed in this study, and can be summarized as follows.

Using Default Environment Factor

The purpose of this task was to compare the site-specific observed capacities to the SIDRA output to determine whether the software predictions are accurate. Using site specific geometry and default SIDRA parameter settings with EF of 1.2 (as would normally be done for any SIDRA analysis), approach lane capacities were generated for a given circulating flow data-point. The field-observed capacities were compared to the SIDRA-derived capacities and the corresponding Root Mean Squared Error (RMSE) calculated.

Optimizing Environment Factor to Reflect Field Conditions

The purpose of this task was to determine the EF value that would generate corresponding approach-lane capacities that will most closely match the field-observed capacities. Following on from the previous task, the default EF was continuously adjusted until the least RMSE value was obtained when comparing the corresponding generated approach-lane capacities to the field-observed values. This way, a more appropriate EF value is obtained that will yield more realistic capacities for Louisiana roundabouts.

Generation of SIDRA Delays and Queues

The EF, resulting in the least RMSE when generated approach-lane capacities are compared to field-observed capacities, was further used to generate corresponding approach lane average delays, level of service (LOS), and 95% back of queue lengths. The average delay generated from the SIDRA analysis did not include geometric delay but rather relied on the HCM delay formula option to allow for direct comparison with the "field-observed delays," which were estimated theoretically using the HCM delay formula as in Equation [13]. Approach LOS values were based on average delay for all movements from an approach and a ratio of the volume to capacity. The 95^{th} percentile queue was the resulting generated queues from the approach delay and was computed as number of vehicles and queue distance.

Discussion of Results

A primary objective of this study was to evaluate driver behavior patterns at roundabouts in Louisiana to calibrate capacity models. In conformance with the state-of-practice, the selected measures of driver behaviors utilized were follow-up times and critical headways, both of which correlate with driver hesitation or aggression as well as familiarity with navigating roundabouts. The following sections discuss the obtained results using the methodologies discussed in the previous chapter.

Study Sites

To ensure the most reliable results, the study adopted strict requirements for selecting sites for the study. Seventeen roundabouts with 41 approaches were initially considered—only these sites/roundabouts met all the criteria listed in the previous section. Data was collected from all these approaches, but for each gap acceptance parameter estimation (follow-up and critical headway), some of these approaches failed to provide qualifying data required for the respective parameter estimation. Thus, both follow-up time and critical headway estimation used data from only 35 approaches from 15 roundabouts.

For capacity and SIDRA analysis, 20 approaches from nine different roundabouts were selected from the list of 17 roundabouts with 41 approaches. The roundabouts were selected such that at least one of the approaches experienced congestion or queues at least for 15 minutes during any time of a day.

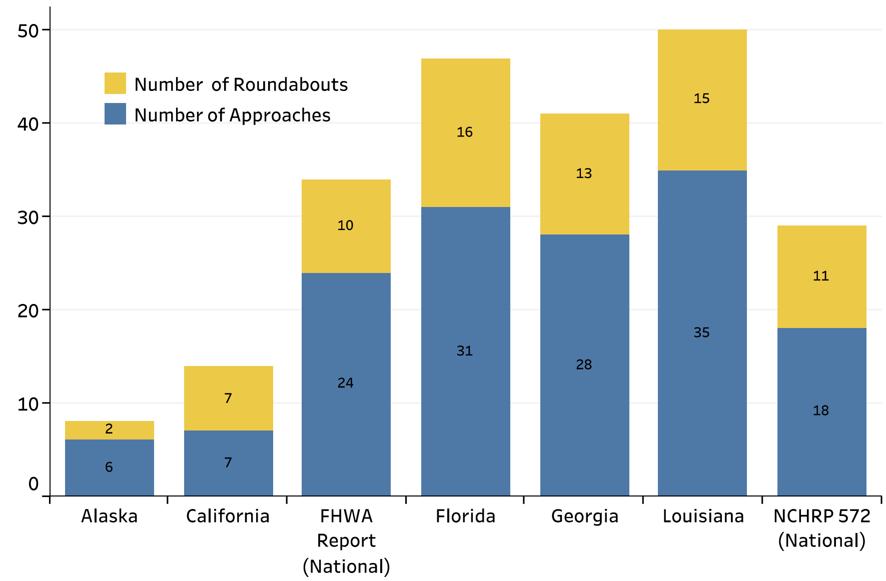
Table 9 summarizes the sites and respective contributions to the study. For example, at site or roundabout R3, data were collected from all the approaches defined by their direction as shown in the table. Age in the table indicates the time the roundabout was open to traffic from the data collection date, which resulted in an average of 8.43 years with a standard deviation of 3.86 years. During the analysis, all approaches were used for the follow-up and critical headway estimation, while only the south approach (S) was used for capacity and SIDRA analysis.

Table 9. Breakdown of sites considered and respective use

Sites	Age, in years	Number of Approaches	Data Collection	Follow-Up and Critical Headway	Capacity Analysis	SIDRA Analysis
R3	11	4	N-E-S-W	N-E-S-W	S	S
R4	11	4	N-E-S-W	N-E-S-W	N-S-W	N-S-W
R7	9	3	N-E-W	N-E-W	N-E	N-E
R8	16	4	N-E-S-W	N-E-S-W	N-E-S-W	N-E-S-W
R10	N/A	2	NE-NW	NE-NW	-	-
R11	N/A	2	E-W	-	-	-
R13	N/A	2	E-S	E-S	-	-
R16	9	1	S	S	S	S
R17	9	1	S	S	-	-
R18	9	1	W	W	-	-
R19	8	2	NW-S	NW-S	NW-S	NW-S
R22	6	2	E-W	E-W	E-W	E-W
R23	9	1	W	W	-	-
R24	12	4	N-E-SE-W	N-E-SE-W	N-E-SE-W	N-E-SE-W
R25	6	2	E-W	E-W	E	E
R26	2	4	N-E-S-W	-	-	-
R27	1	2	N-E	N-E	-	-
Total = 17	Mean = 8.43 years, SD = 3.86	41	41	15 roundabouts and 35 approaches	9 roundabouts and 20 approaches	9 roundabouts and 20 approaches
	N - North, E - East, S - South, W - West, NE - North East, NW - North West,					

Relative to similar studies conducted in the United States, this study considered the highest number of sites and approaches. Figure 11 shows the number of sites and approaches used for gap acceptance parameter estimation for this study and others. Amongst the other studies considered, the Alaska study used both the least number of sites and roundabouts—six approaches from two roundabouts were assessed. The Louisiana study assessed the highest number of approaches and roundabouts. Most of these studies were conducted for statewide applications. However, the NCHRP 572 and FHWA Report were nationally conducted studies that were used to develop the HCM 2010 and HCM 6 models, respectively. The former only assessed 18 approaches from 11 roundabouts, and the latter, 24 approaches from 10 roundabouts for single-lane roundabouts.

Figure 11. Similar studies and the number of roundabouts and approaches considered



Follow-Up Times

As earlier defined, the follow-up time (t_f) is the time difference between successive vehicles departing the yield line and accepting the same gap in circulatory flow. For all the approaches that provided data meeting both conditions, the number of qualifying observations, the mean follow-up time for the site, and the standard deviation were recorded—not every site where data was collected provided qualifying data for follow-up time estimation. As stated earlier, only 35 approaches from 15 roundabouts qualified for this assessment. To assess how follow-up times varied on different classes of roads, the data were classified according to functional classes to determine the mean t_f and standard deviation (SD) for each functional class. Further, DOTD districts were also used as classes to measure how t_f varied across districts in Louisiana. It is important to note that the effect of exiting vehicles was not considered in evaluating follow-up times.

From the several hours of footage taken from each site, 117, 558 events meeting the conditions for follow-up times were observed. These ranged between 2.46 seconds and 3.76 seconds, weighted average of 3.36 seconds with a standard deviation of 1.23. Table 10 shows the descriptive statistics for all data considered. The summary of the result per site is included in Appendix C.

Table 10. Descriptive statistics of follow-up times

Description	Value (s)
Total Number of observations	117, 558
Weighted Mean, s	3.36
Standard Deviation	1.23
Minimum	2.46
Maximum	3.76

Figure 12 shows results for each roundabout.

Figure 12. Selected roundabouts with number of observations of t_f and the mean

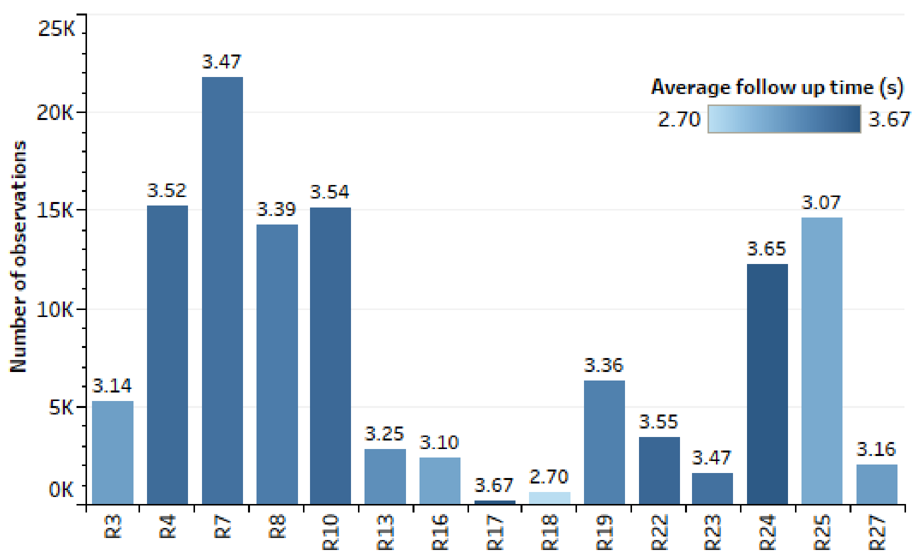


Table 11 shows results based on the functional class of the site. For the functional classes, the majority of the observations, 98,273 were collected from arterials. The mean of these was 3.35, 0.3% lower than the Louisiana average. Only two approaches were local roads, thus providing the least number of qualifying observations. The mean was 3.38, which is 0.6% larger than the Louisiana average.

Table 11. Follow-up times per functional class

Functional Class	Number of Approaches	Number of Observations	Mean	SD
Locals	2	4,049	3.38	1.20
Collectors	15	15,236	3.39	1.39
Arterials	18	98,273	3.35	1.21

Table 12 shows results based on the DOTD district. It is observed Lafayette provided the highest number of total observations in agreement with the prior assertion that Lafayette having the most roundabouts (contributing 21 approaches) was most likely to provide the highest number of observations. The mean t_f for Lafayette was 3.39, which is a value 0.9% higher than the Louisiana average. With only five approaches, East Baton Rouge had the least number of qualifying observations and an average t_f of 3.25, also 3% lower than the Louisiana average.

Table 12. Follow-up times per DOTD district with qualifying data

DOTD District (No)	Number of Approaches	Number of Observations	Mean	SD
Lafayette Parish (3)	21	76,362	3.39	1.26
East Baton Rouge (61)	5	9,314	3.25	1.21
St. Tammany (62)	9	31,882	3.29	1.15

Critical Headway

As discussed in the previous chapter, the critical headway is the least gap in circulatory flow that an approach driver is willing to accept. Therefore, the critical headway is ideally greater than the largest rejected headway but less than all accepted headways. For this study, similar to the case of follow-up time estimation, qualifying events for critical headway estimation were obtained from only 35 approaches representing 15 of the studied roundabouts. These critical headways ranged between 3.94 and 6.89 seconds as shown in Table 13 with a weighted average of 4.76 seconds and a standard deviation of 0.54. A total of 9,245 observations were recorded. The specific details have been included in Appendix D.

The roundabout approach R25 West, an arterial located in St. Tammany, had the largest critical headway of 6.89. The least of 3.94 was recorded at R7 North, an arterial located in Lafayette. The least number of qualifying observations were obtained from approach R27 East, a collector in Lafayette, and the highest was R7 East, an arterial in Lafayette.

Table 13. Descriptive statistics of critical headway

Description	Value (s)
Total Number of observations	9,245
Weighted Mean, s	4.76

Description	Value (s)
Standard Deviation	0.54
Minimum	3.94
Maximum	6.89

Considering results per roundabout as displayed in Figure 13, a site located in Lafayette, R8 provided the highest number of observations, closely followed by another roundabout also in Lafayette, R4 with the critical headway of 4.54s. Roundabout R17 located in East Baton Rouge, again provided the least number of observations. Roundabouts R17 and R18 recorded the highest critical headway of 6.72s.

Figure 13. Selected roundabouts with a number of observations of t_c and the mean

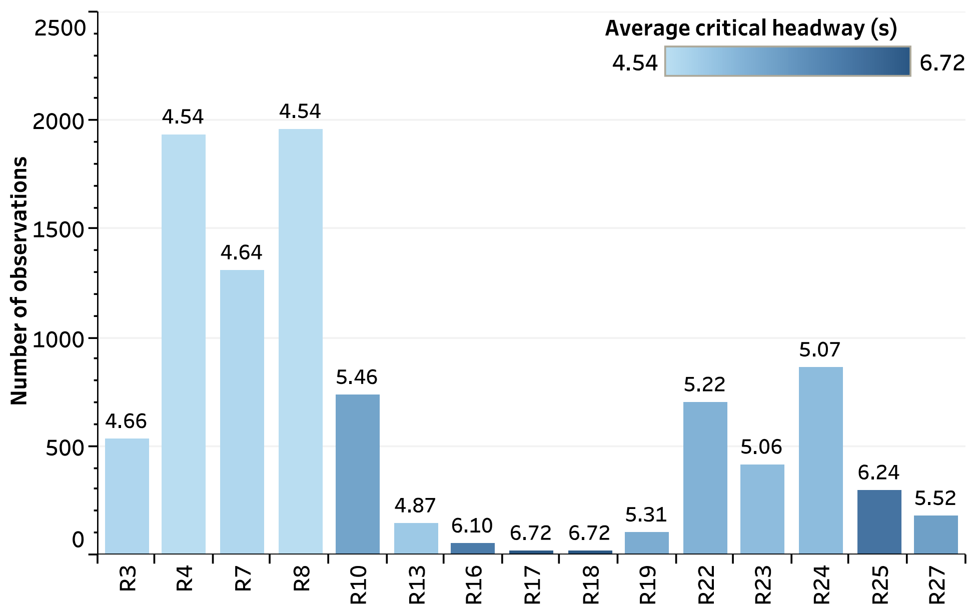


Table 14 shows results based on the functional class of the site. Considering functional classes, both collectors and arterials showed results close to the Louisiana average of 4.76—the former produced an average of 4.71 and the latter 4.70. Local approaches showed an average critical headway of 5.54, which is 16.4% higher than the Louisiana average.

Table 15 shows results based on DOTD districts. Lafayette had the least t_c of 4.54 seconds, followed by St. Tammany with 5.25 seconds and East Baton Rouge with a high t_c of 5.67 seconds.

Table 14. Critical times per functional class

Functional Class	Number of Approaches	Number of Observations	Mean	SD
Locals	2	562	5.54	1.65
Collectors	15	3640	4.71	0.95
Arterials	18	5043	4.70	0.99

Table 15. Critical times per DOTD district with qualifying data

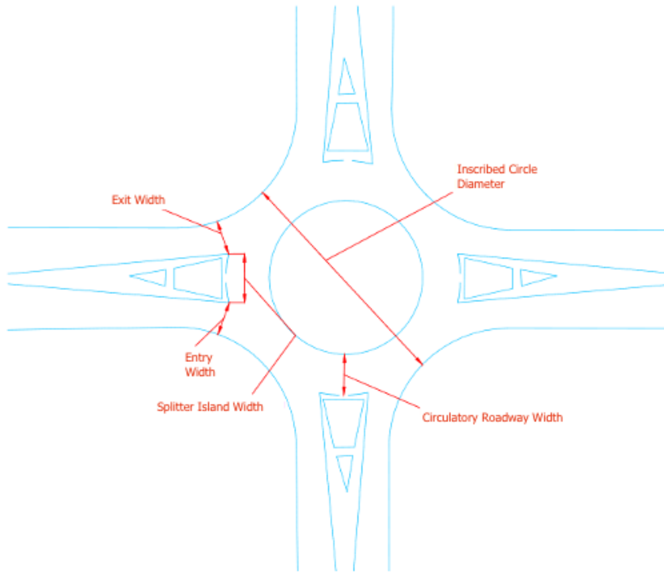
DOTD District	Number of Approaches	Number of Observations	Mean	SD
Lafayette Parish	21	6,787	4.54	0.90
East Baton Rouge	5	181	5.67	1.49
St. Tammany	9	2,277	5.25	1.30

Considering results across clusters, the only consistency is noticed in the arterials. These had the best case of potential performance with the least critical headway and follow-up time, both of which are proven to be inversely proportional with capacity performance or throughput. This suggests driver performance in Louisiana is best along arterials. Given that arterials are relatively high-speed roads, it would explain why drivers accept the least gaps, and maintain the least follow-up times on such roads to keep up with the swift/speedy movement characteristic of arterials.

Geometric Analysis

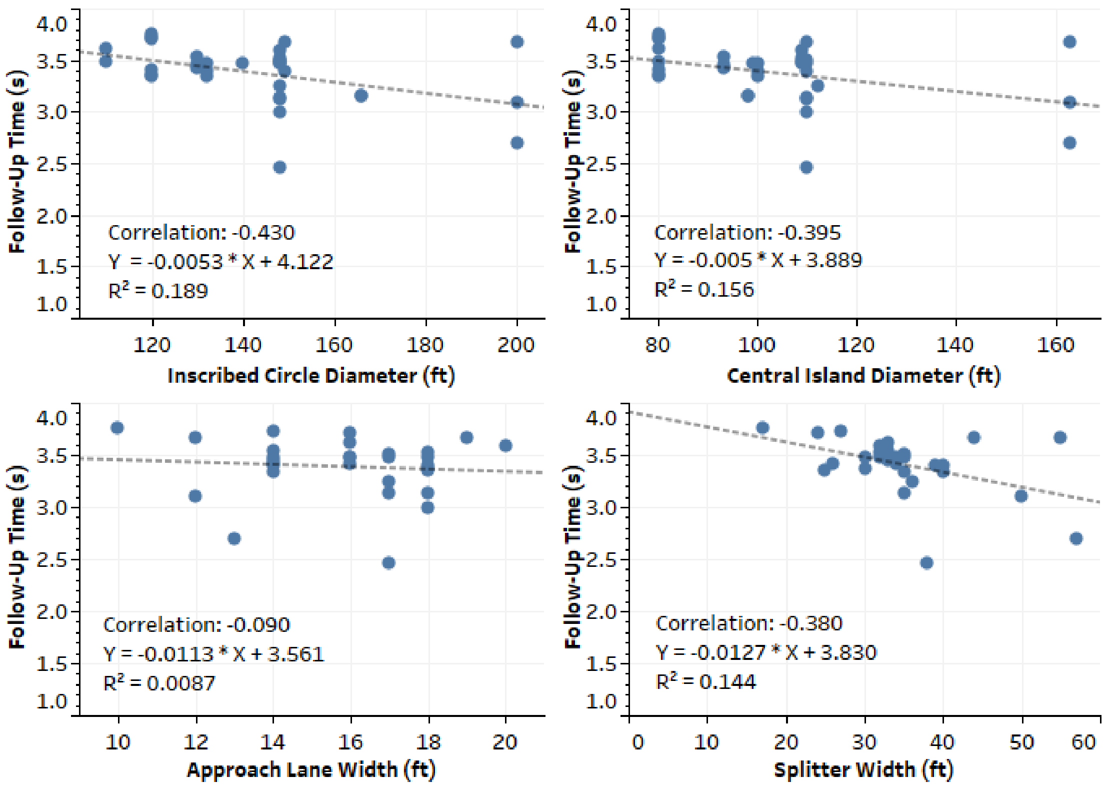
The geometric layout of the roundabout impacts its performance [35]. Minor geometric modifications can impact flow and safety. To determine whether geometric features impact follow-up times, the correlation between follow-up time and the following geometric features were explored: inscribed circle diameter (ICD), splitter (island) width, central island diameter, and approach lane width. These are illustrated in Figure 14.

Figure 14. Geometric quantities considered



The results from these tests are displayed in Figure 15. From the scatter plots in Figure 15, it is observed that each geometric property trends in the expected direction: each is inversely proportional to follow-up time, and by extension, directly proportional to capacity since a lower follow-up time implies higher capacities. However, the values of the correlation and coefficient of correlation (R-squared) show very weak correlations between follow-up times and each geometric feature. It can be seen that ICD showed the highest correlation but, once again, showed a relatively weak coefficient of correlation. Approach lane width least correlates with follow-up times, suggesting that the choice of approach lane widths least affected the observed follow-up times. It can be seen visually from Figure 15 that, for a given geometric feature value, multiple follow-up times exist. This implies that variability among the different sites, probably derived from driving behavior, has a more dominant effect than the geometric feature being explored. For this reason, geometric features are not considered in the development of the local capacity model.

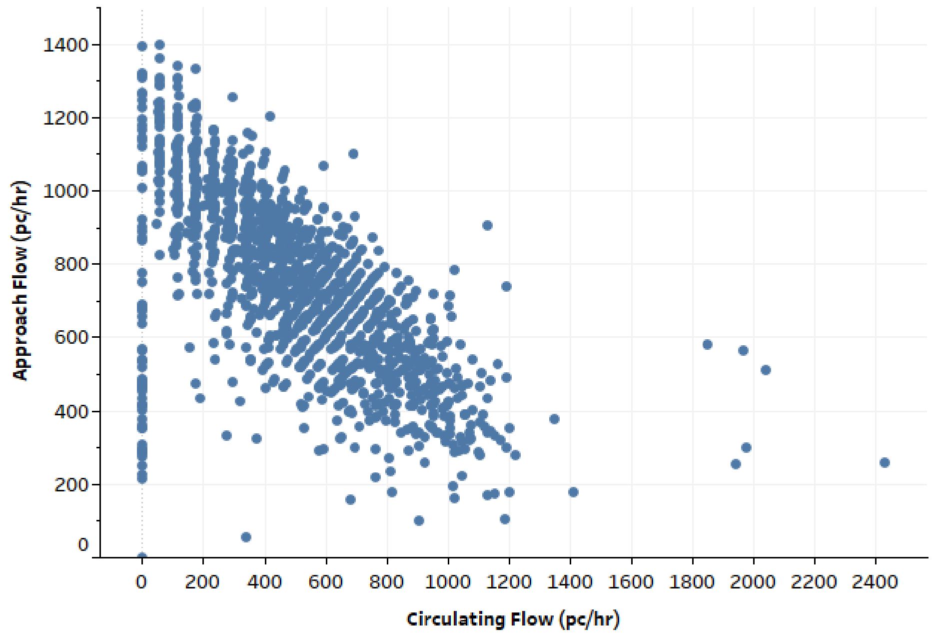
Figure 15. Scatter plot of follow-up time v. ICD and CID



Capacity Analysis and Model Development

For each one-minute bin, the volume counts were obtained for both approach/entry flows and circulatory flows, both in vehicles per hour. Using the former as y-values and the latter as x-values, a scatter plot of 1,696 points was created for the raw data as shown in Figure 16. This visually explains how the approach flow correlates with the circulatory flow. A relatively strong negative correlation is evident from the plot, where moving along the X-axis, lower values of Y are seen and vice versa—for a lower circulatory flow, one can expect a higher approach flow.

Figure 16. Scatter plot of all field-observed data



Assessment of HCM 2010 Models

Capacity models estimate the capacity of roundabouts based on the circulatory flow (the dependent variable). Using the HCM 2010's capacity model for single-lane roundabouts, Equation [6], and the field-observed circulatory flows, the capacity in passenger cars equivalent per hour was obtained. Similar to the NCHRP Report 572, this output was compared to the field-observed capacities (approach volumes) using the root-mean-square errors (RMSE). The RMSE is the standard deviation of how far from the regression line the data points are or how close to the regression line observed points lie. A low RMSE value suggests all points lie very close to the regression line, and a high RMSE suggests points lie farther from the regression line. A good model results in a low RMSE—points lie close to the regression line.

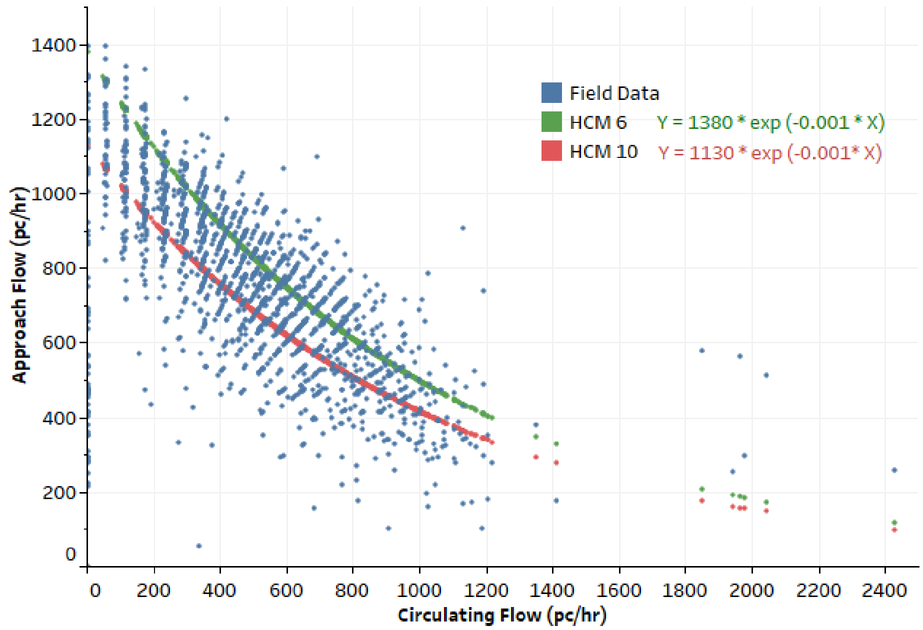
Also using the HCM 6's model for single-lane roundabouts, the above procedure was repeated, and the output compared to the field observations in a similar manner. With reference to Table 16, the RMSE for the HCM 2010 to the field data is 186.12, and that of the HCM 6 model is 227.17. Based on the resulting RMSEs, it is evident that the HCM 2010 model is a better predictor of performance for Louisiana conditions than the HCM 6 model. This suggests that the data used to develop the HCM 2010 model produced similar gap acceptance parameters, relative to the HCM 6's to our field observations, thus the lower RMSE or better fitness of the HCM 2010's capacity model.

Table 16. Summary of HCM models' performance

Model	**Source**	**RMSE computed with field-observed capacity**
$c_{e,pce} = 1{,}130e^{(-1.0\times10^{-3})v_{c,pce}}$	HCM 2010	186.12
$c_{e,pce} = 1{,}380e^{(-1.02\times10^{-3})v_{c,pce}}$	HCM 6	227.17

In Figure 17, the capacity model from both HCM versions are plotted against the field observations. The field-observed measures (blue points) are plotted by locating the point of intersection between two perpendicular lines drawn from an observed circulatory flow (on the X-axis) and its corresponding approach flow (on the Y-axis) for the same bin. All 1,696 bins are each represented by a blue point denoting the point of intersection created by the two perpendicular lines drawn through the observed flows for each bin. The HCM 2010 model is shown in red, and the HCM 6 model shown in green. These lines are developed using the equations in Table 16 and using the field-observed circulatory flows ($V_{c,\ pce}$) to compute the corresponding approach flows. The lines are drawn along the points created from the intersection of perpendicular lines from the field-observed circulatory flows, and the resulting approach flows generated by the respective HCM model. A visual inspection suggests that the HCM 2010 is a better predictor as more points fit closely to its line relative to the HCM 6 model. The field-observed data (blue points) lie closer to the red line—a relatively lower dispersion is noticed. The same data has a wider spread or dispersion around the green lie, with most of them lying farther below the green line.

Figure 17. Scatter plot of field data superimposed with HCM models



Exploration and Calibration of Models

The measures of driver behavior evaluated were both follow-up time (t_f) and critical headway (t_c). Similar to the NCHRP 572's method and the HCM's calibration recommendations, the two field observable parameters; follow-up time, t_f; and critical headway, t_c are applied to the form of the regression models in Equation [11] to ensure calibration to local conditions. The t_f and t_c relate to the general form of the regression models in Equation [11] as explained early on:

$$Q = Xe^{(Y)v_{c,pce}} \quad [11]$$

where,
Q = Approach lane capacity, adjusted for heavy vehicles (pc/h),
X = 3600/t_f,
Y = (t_c-0.5t_f)/3600, and
$V_{c,\,pce}$ = circulating flow rate (pc/h).

The y-intercept of the trend equation (X in Equation [11]) represents the approach's saturation flow or the highest possible approach lane capacity, also known to be the highest throughput for an approach lane with queueing but with zero opposing flow or circulatory flow. This was used to set the intercept before the trend line was generated for the locally calibrated model.

Using a t_f of 3.36 seconds and a t_c of 4.76 seconds, the parameters for the fitted equation were obtained and presented as Equation [16]:

$X = 3600/3.36 = 1072.3$

$Y = (4.76\text{-}0.5*3.36)/3600 = 0.0009$

$$Q = 1072.3e^{(-0.0009)v_{c,pce}} \qquad [16]$$

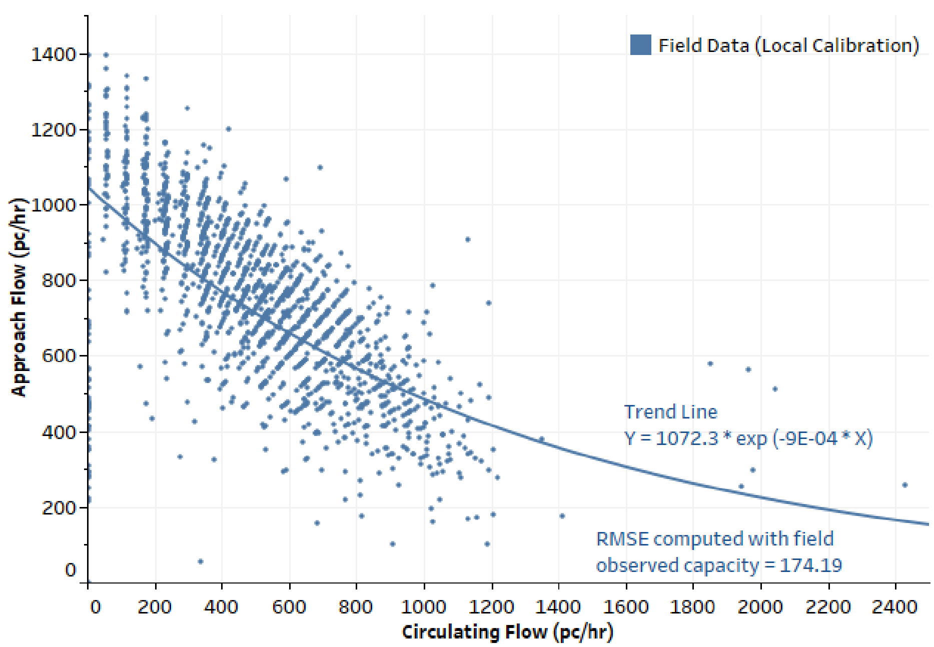
where,

$Q = C_{pce}$= Approach capacity in passenger car equivalent per hour, and

$V_{c,\, pce}$ = Circulatory volume in passenger car equivalent per hour.

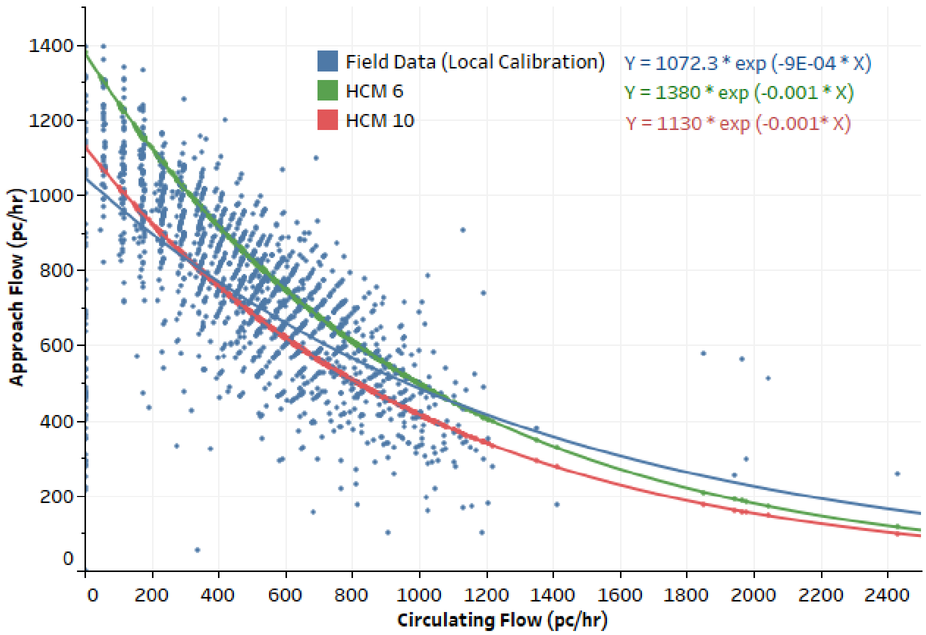
Figure 18 shows the fitted trend line imposed on the field observations and shows the goodness of fit of the model in terms of RMSE as 174.19.

Figure 18. Locally calibrated model with field-observed data



The calibrated model in comparison with the field-observed data resulted in an RMSE of 174.19. Figure 19 combines the locally calibrated model with the HCM models to visually express their comparative performance.

Figure 19. Field data with HCM models and locally calibrated model



In total, three models were assessed: HCM 2010, HCM 6, and the regression models calibrated to the follow-up time and the calibrated model specific to both t_f and t_c. Table 17 summarizes these models. From Table 17, it is evident that Model 3, the calibrated model, is the best fit showing the least RMSE value. Relatively, Model 2, the HCM 6 model, has the largest RMSE, thus being the least predictive model considered.

Table 17. Summary of all models

Model No.	Model	Source	RMSE
1	$c_{e,pce} = 1{,}130e^{(-1.0\times10^{-3})v_{c,pce}}$	HCM 2010	186.12
2	$c_{e,pce} = 1{,}380e^{(-1.02\times10^{-3})v_{c,pce}}$	HCM 6	227.17
3	$c_{e,pce} = 1072.3e^{(-0.0009)v_{c,pce}}$	Locally calibrated model	174.19

Estimation of Delays, Queue Length, and Level of Service

Peak Times

To support LOS analysis, worse case conditions were required. These worse case conditions are caused when a facility reaches its capacity, often during peak travel times.

For this study, peak hours were fixed at 7AM to 9AM for morning peaks and 4PM to 6PM for afternoon peaks. Each 15-minute period within the identified peak hours were used for volume counts for approach (V_a) and circulatory volumes (V_c) to determine peak flows. The procedure outlined in Chapter 22 of the HCM 6 was closely followed. The delay analysis was conducted for 20 approaches from 9 roundabouts that provided qualifying data for the model development and calibration.

i. For each 15-minute period, (e.g., 7:00AM to 7:15AM, 7:15AM to 7:30AM, etc.), the total vehicles observed approaching the roundabout, V_a were counted in vehicles per hour (vph), and the corresponding vehicles observed in the circulatory path that conflicted the approach path, V_c, were also counted in vph.
ii. The sum of V_a and V_c is computed as V_t for each period.
iii. The maximum V_t value for all days of data collection was selected and the corresponding period used as the peak 15-minute flow.

Delays, queue length, and LOS estimation

The steps followed to determine the delays are outlined below.

i. The V_c and V_a for the identified peak 15-minute period are both multiplied by 4 to obtain an hourly peak flow in vehicles per hour.
ii. To make provision for observed large vehicles, a heavy vehicles factor, f_{hv}, is computed using Equations [17] and [18] below and volumes are converted into demand flow rates.

$$v_{i,pce} = \frac{v_i}{f_{hv}} \tag{17}$$

$$f_{hv} = \frac{1}{1+P_T(E_T-1)} \tag{18}$$

where,
$v_{i,pce}$= demand flow rate for movement i (pcu/h),
v_i = demand flow rate for movement i (veh/h),
f_{hv} =heavy-vehicle adjustment factor,
P_T = Proportion of demand volume that consist of heavy vehicles, and
E_T = Passenger car equivalent for heavy vehicles.

iii. Using the capacity equation developed through local calibration (Table 17, Model 3), the capacity is computed based on the demand volume (V_c).

iv. The computed capacity from iii is converted back to vehicles per hour by multiplying by the F_{hv} computed in ii.

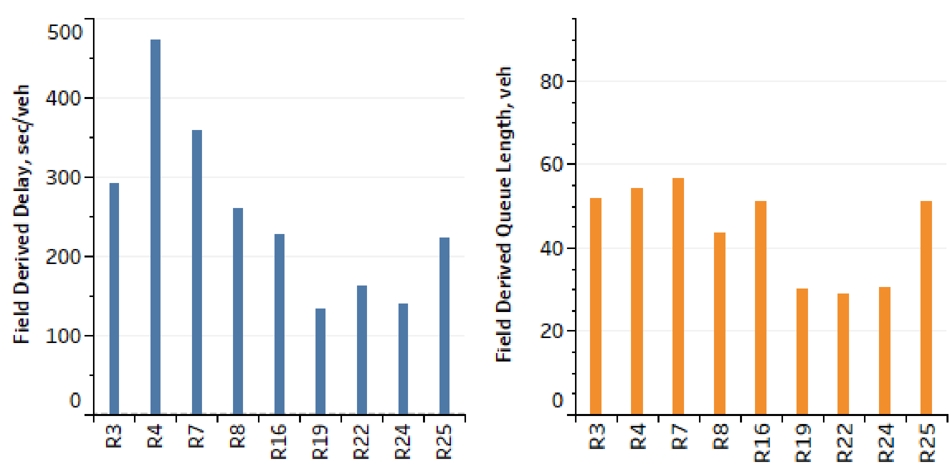
v. The volume to capacity ratio is calculated by dividing the approach volume obtained in ii by the approach capacity in iv.

vi. The control delay is computed using Equation [13].

vii. The 95th percentile queue length is estimated using Equation [15].

Figure 20 shows the result summarized at each roundabout location. It shows both average field derived delays and 95th percentile queue lengths estimated at each approach, using Equation [13] and Equation [15], respectively. Using Table 3, the corresponding LOS is determined for each approach. The detailed result is provided in Appendix F.

Figure 20. Average field derived delays and 95th percentile queue length



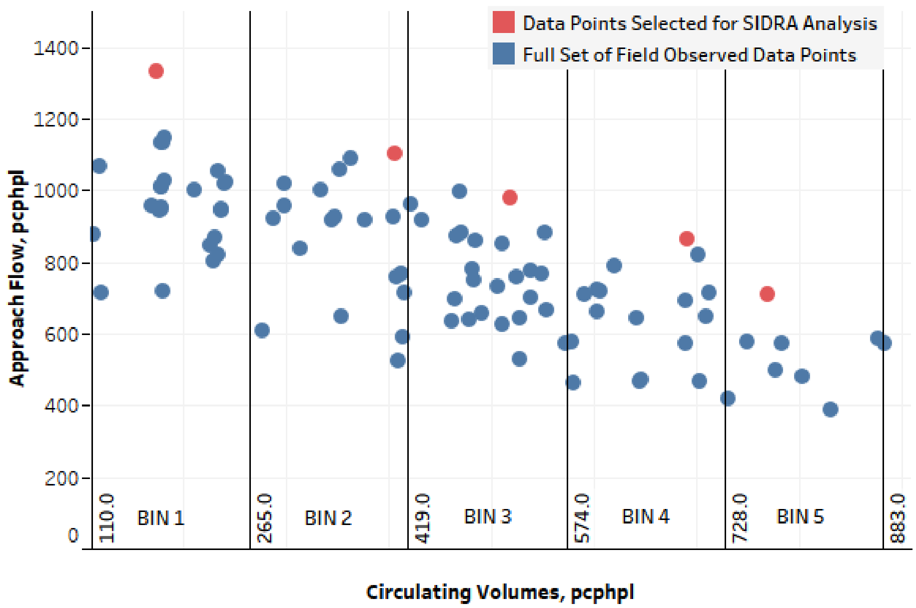
SIDRA Output

Establishing Base RMSEs

Table 9 shows the list of sites and corresponding approaches used for the SIDRA analysis. The study used a total of 20 approaches from 9 different roundabouts for the

SIDRA analysis. The same locations were used for the delay, queue length, and LOS calculations. Each roundabout approach observed a wide range of approach volume capacities that corresponded to specific circulating volumes as shown in Figure 15. The data was sorted and separated into five bins based on the circulating flow measurement. This consisted of both saturated and unsaturated flows and the bins were constructed such that representative flows can be selected across varying traffic conditions. Within each of the five bins, the data point with the highest total (circulating plus approach) flow was then selected for analysis. Figure 21 demonstrates an example for one of the approach lanes and shows the five bins and the selected data pair per bin that were finally chosen for analysis.

Figure 21. Example of bin partition and selection of SIDRA data points



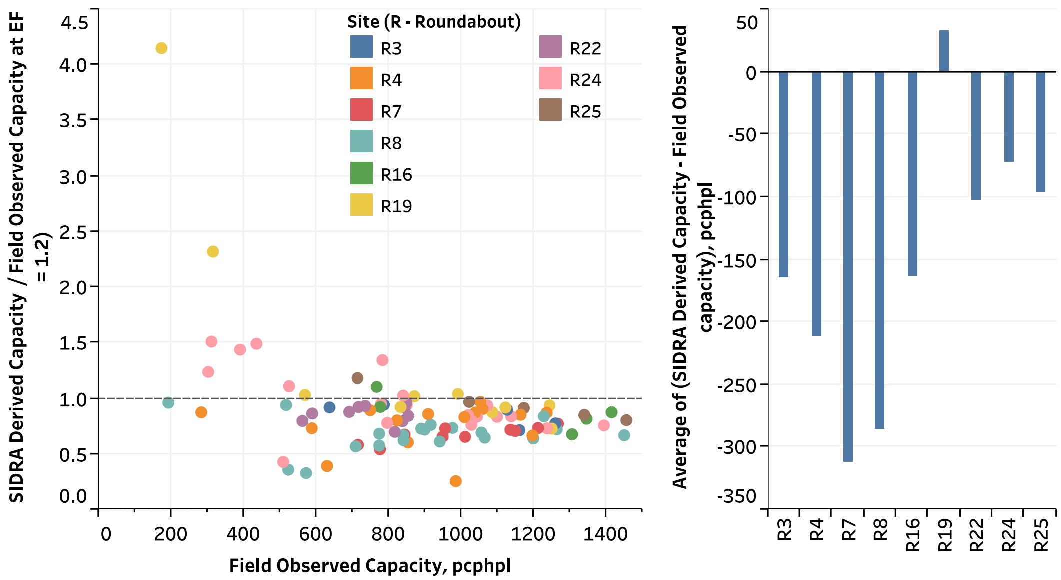
Eventually, five pairs of flows were analyzed in SIDRA for each of the 20 approaches from nine roundabouts, resulting in a total of 100 analyses. Each analysis resulted in a generation of an approach capacity, using the selected circulating volume and EF of 1.2, and this was then compared to the field-observed approach capacity corresponding to the same circulating volume. Table 18 shows an excerpt of a SIDRA output for one site. In this example, South, East, North, and West are the approach legs of a roundabout. The demand flows correspond to the SIDRA generated approach volumes for the selected approach, and the output also shows the corresponding average delay, LOS, and 95% back of queue length.

Table 18. Example of SIDRA output

Movement Performance - Vehicles												
Mov ID	Turn	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	Distance ft	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed mph
South: RoadName												
3	L2	1	3.0	1.112	82.8	LOS F	75.6	1934.4	1.00	2.61	4.73	16.1
8	T1	1126	3.0	1.112	82.8	LOS F	75.6	1934.4	1.00	2.61	4.73	16.1
18	R2	1	3.0	1.112	82.8	LOS F	75.6	1934.4	1.00	2.61	4.73	15.8
Approach		1128	3.0	1.112	82.8	LOS F	75.6	1934.4	1.00	2.61	4.73	16.1
East: RoadName												
1	L2	1	3.0	0.009	9.9	LOS A	0.1	1.4	0.87	0.62	0.87	32.2
6	T1	1	3.0	0.009	9.9	LOS A	0.1	1.4	0.87	0.62	0.87	32.1
16	R2	1	3.0	0.009	9.9	LOS A	0.1	1.4	0.87	0.62	0.87	31.1
Approach		3	3.0	0.009	9.9	LOS A	0.1	1.4	0.87	0.62	0.87	31.8
North: RoadName												
7	L2	1	3.0	0.003	3.0	LOS A	0.0	0.3	0.03	0.00	0.03	35.6
4	T1	1	3.0	0.003	3.0	LOS A	0.0	0.3	0.03	0.00	0.03	35.5
14	R2	1	3.0	0.003	3.0	LOS A	0.0	0.3	0.03	0.00	0.03	34.4
Approach		3	3.0	0.003	3.0	LOS A	0.0	0.3	0.03	0.00	0.03	35.2
West: RoadName												
5	L2	1	3.0	0.258	5.2	LOS A	1.1	28.0	0.03	0.00	0.03	35.6
2	T1	313	3.0	0.258	5.2	LOS A	1.1	28.0	0.03	0.00	0.03	35.4
12	R2	1	3.0	0.258	5.2	LOS A	1.1	28.0	0.03	0.00	0.03	34.3
Approach		315	3.0	0.258	5.2	LOS A	1.1	28.0	0.03	0.00	0.03	35.4
All Vehicles		1450	3.0	1.112	65.6	LOS E	75.6	1934.4	0.79	2.03	3.69	18.3

Likewise, outputs were obtained for all 100 data points and the demand flows recorded and compared to their corresponding field-observed capacities. Figure 22 shows the scatter plot for both capacities—ratio of demand flows from SIDRA output to field-observed capacities versus only field-observed capacities—at an EF of 1.2. The bar plot in the figure shows the average of the difference between the two capacities for each of the 9 roundabouts. Overall, the figure shows discrepancies between field-observed and SIDRA-derived capacities with SIDRA underestimating the field-observed capacities at most of the approaches. It implies software-derived capacities were lower than the field-observed capacities. An average difference of minus 162.675 pcphpl was estimated between SIDRA capacities and field-observed capacities. Furthermore, the comparison resulted in an RMSE of 247.82.

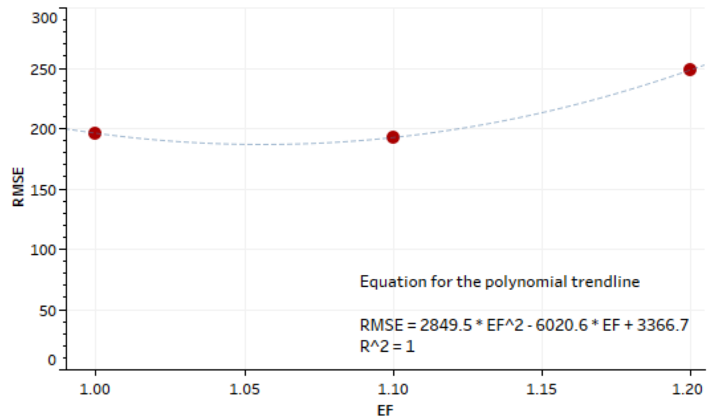
Figure 22. Comparison of field-observed and SIDRA-derived capacities at EF of 1.2



Determining Louisiana Appropriate Environment Factor

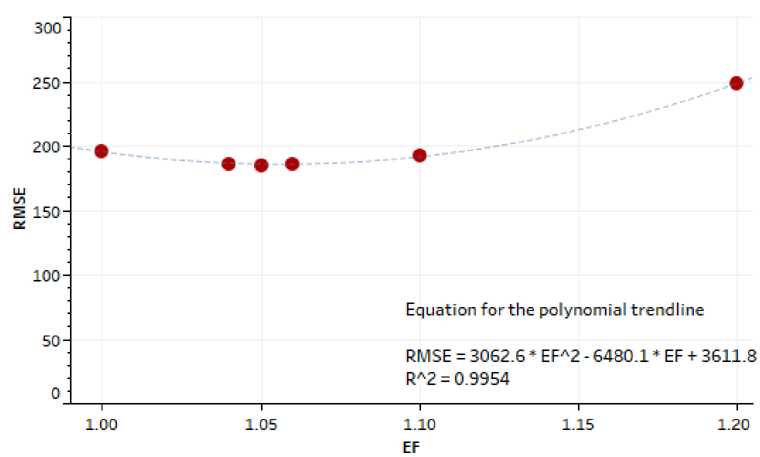
Initial EF of 1.2 (default value), 1.1, and 1.0 were assumed and used to generate SIDRA-derived capacities as previously. With each EF, a corresponding RMSE was determined between the field-observed capacities and their corresponding SIDRA-derived capacities. Figure 23 shows a 2nd order polynomial trend line that was fitted for the curve of RMSE versus EF and resulted in an EF of between 1.05 and 1.06 as the most appropriate value. Capacities generated at different EFs are summarized in Appendix E.

Figure 23. Initial plot of RMSE versus environmental factor (EF)



To be able to determine which value of EF generates the least RMSE, additional analysis was undertaken to include EF of 1.05 and 1.06 in the generation of SIDRA-derived capacities. Once again, the RMSE corresponding to the comparison of the SIDRA-derived capacities to the field-observed capacities were computed. A graph of RMSE versus EF was developed again for all five values of EF, i.e., 1.0, 1.1, 1.04, 1.05, 1.06, and 1.2. A trend line was generated as previously to generate a polynomial equation equating RMSE to EF. Figure 24 shows the final graph obtained. The circulating flows corresponding to field-observed capacities, as well as the SIDRA-derived capacities for each of the EF analyzed for each approach has been tabled and presented in Appendix E.

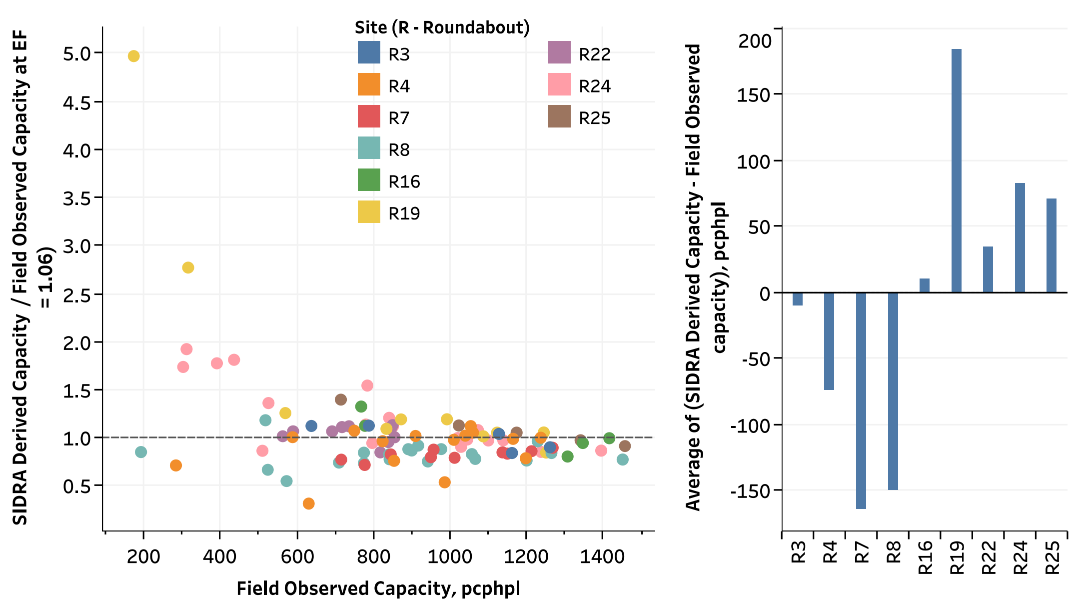
Figure 24. Final plot of RMSE versus EF



Based on the analyses, it appears that EF of 1.06 yielded the lowest RMSE for the given dataset. This implies that contrary to the default EF value of 1.2 currently being used in SIDRA assessments, an EF value of 1.06 generates approach capacities which more accurately reflect field conditions. Following, the EF of 1.06 was used to generate SIDRA-derived capacities for all the 100 data points representing all nine roundabouts, and these were compared against the actual field-observed capacities.

Figure 25 shows the plot of these two sets of approach capacities, similar to Figure 22, but for an EF of 1.06 as opposed to 1.2. The scatter plot shows the SIDRA-derived capacities show better match to the field-observed capacities at EF of 1.06 with the ratio of both the capacities at most of the approaches aligning to the 1, where 1 shows the exact match. Once again, the bar plot shows the differences between the average capacities observed at each roundabout. Compared to an average difference of minus 162.675 pcphpl at EF of 1.2, an average of just minus 15.559 was estimated at EF of 1.06. An average of the absolute difference, as shown in the bar plot, was estimated as 142.132 pcphpl compared to 206.863 pcphpl at EF of 1.2. This showed that the EF of 1.06 generated capacities which were closest to field-observed capacities, and hence, should be preferable over EF of 1.2.

Figure 25. Comparison of field-observed and SIDRA-derived capacities at EF of 1.06

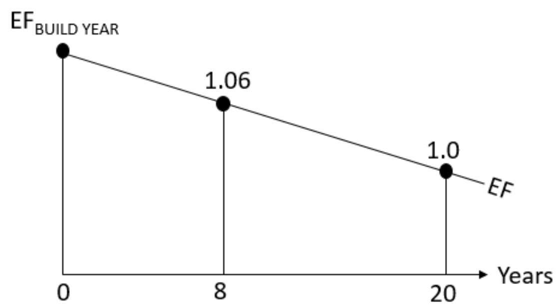


However, from Table 9, the roundabouts evaluated for EF have an approximate average age of eight years. Literature [20] suggests a Design Year (of 20 years) EF of 1.0 for single-lane roundabouts in the US. Since the objective is to determine an appropriate EF for the Build Year (of zero years), Figure 26 was developed to extrapolate and compute an appropriate Build Year EF as in Equation [19]:

$$EF_{Build\ Year} = 1.06 - \left[\frac{(8-0)}{(20-8)}\right](1.0 - 1.06) = 1.10 \qquad [19]$$

The above analysis suggests a Build Year EF of 1.1 will probably be more reflective of Louisiana driver behavior. This is based on the assumption that driving behavior at roundabouts improves linearly over time, from the Build Year to the Design Year.

Figure 26. Extrapolation of EF to Build Year



Comparison of Delay, Queue Length, and Level of Service

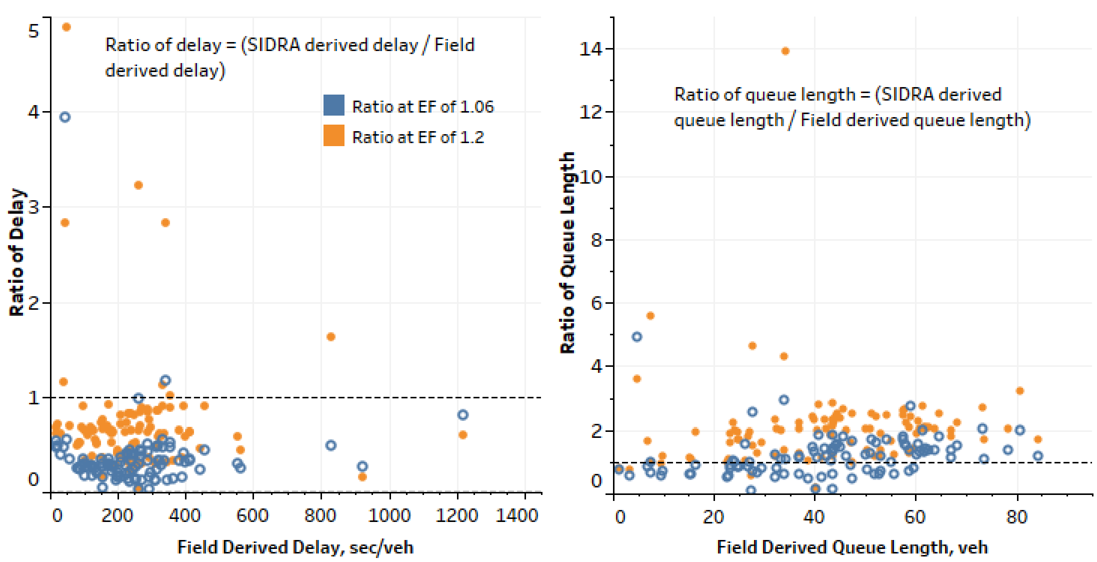
Next, the study compared delay, queue length and level of service (LOS) for both field derived and SIDRA analysis for currently used EF of 1.2 in addition to EF of 1.06. It is worth noting that the field observations are referred to as field-derived values because they were determined theoretically using HCM formula as shown in Equation [13], Equation [15] and Table 8, respectively. The SIDRA-generated components were read directly off the SIDRA outputs, as shown in the example in Table 18, but for EF of 1.2 and 1.06. Appendix F shows the field corresponding field-derived delays, queue lengths, and LOS as well as the corresponding SIDRA delays, queue lengths, and LOS at EF of 1.2 and 1.06.

Figure 27 shows the scatter plot of the ratio of SIDRA-generated delays and queue lengths to the field-derived delays and queue lengths which corresponds to the field observations. Ideally, the ratio of two estimates aligning to one will be preferred, as that will indicate that the two sets of data being compared are perfectly similar. The ratio of two estimates at EF of 1.2 is close to 1 compared to a ratio of two estimates at EF of 1.06. This implies that SIDRA-generated delays at EF of 1.2 is close to field-derived delays.

Furthermore, this observation is supported by computing RMSE between each pair of data point between the SIDRA-generated delays and field-derived delays. The result shows RMSE of 199.39 and 231.31 sec per for EF of 1.2 at EF of 1.06, respectively. It confirms that EF of 1.2 generated delays which were closer to the field-derived delays than EF of 1.06 did. Altogether, both SIDRA-generated delays were lower than the field-derived delays.

Similarly, 95th percentile queue length (number of vehicles) was estimated for various pairs of circulating and approaching volumes using Equation 15. The field-derived queue lengths were then compared to SIDRA-estimated queue lengths at EF of 1.2 and EF of 1.06, respectively. Similar to delay, the ratio of SIDRA-generated queue lengths at EF of 1.2 and 1.06 compared to field-derived queue length shows ratio at EF of 1.06 closer to 1. This implies that SIDRA-generated queue length at EF of 1.2 is close to field-derived delays and queue length.

Figure 27. Comparison of field-derived and SIDRA-estimated delay and queue length



In addition, field-derived LOS and SIDRA-generated LOS for EF of 1.2 and 1.06 were compared. Table 19 shows a summary of the results of the comparison. It shows that

SIDRA-derived delay was close to field-derived delay at EF of 1.2 compared to EF of 1.06. However, SIDRA queue length was found close to field-derived queue length at EF of 1.06.

Table 19. Summary of delay and queue length

	Delay, sec/veh			Queue length, veh		
	Field derived delay	**SIDRA Delay, EF = 1.2**	**SIDRA Delay, EF = 1.06**	**Field derived queue length**	**SIDRA queue length, EF = 1.2**	**SIDRA queue length, EF = 1.06**
Minimum	11.360	7.800	6.200	1.329	1.100	1.000
Maximum	1220.490	1348.800	989.100	84.291	474.100	162.300
Average	253.321	175.508	87.954	42.199	88.788	52.611
Standard Deviation	175.752	192.235	116.276	18.479	62.592	36.683

Table 20 below shows LOS improved at EF of 1.06. It shows the number of approaches that matches their LOS from field-derived estimates to SIDRA analysis at EF of 1.2 and 1.06. The result shows a larger proportion of approaches matching their LOS from field to SIDRA estimates at EF of 1.2 compared to EF of 1.06.

Table 20. Number of approaches with LOS for different volume types

Type	Number of Approaches		Proportion	
	EF = 1.2	**EF = 1.06**	**EF = 1.2**	**EF = 1.06**
Matching Pairs	87	59	87%	59%
Nonmatching Pairs	13	41	13%	41%
Total approaches	100	100	100%	100%

Conclusions

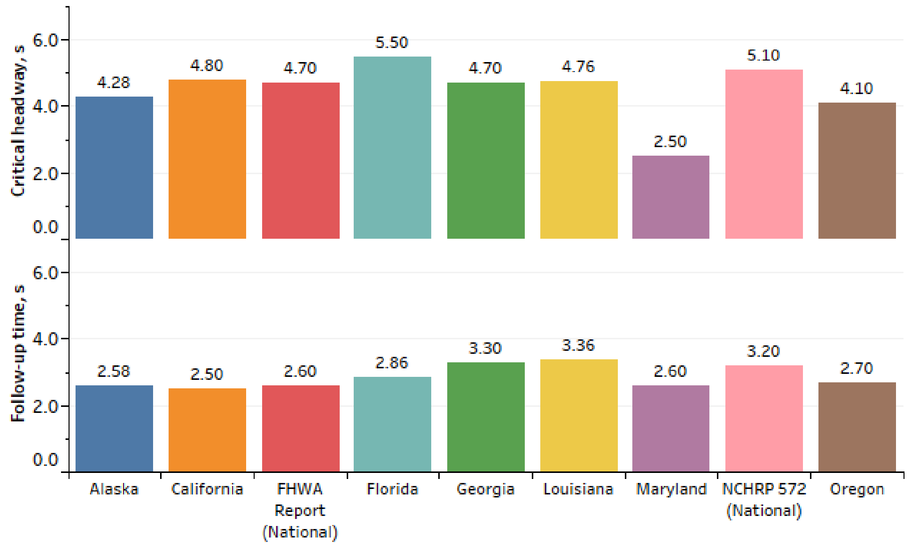
The primary aim of this research was to calibrate roundabout capacity models to Louisiana conditions. The following sections summarize the process and compares some of the findings to results from other regions in the United States.

Gap Acceptance Parameters

The gap acceptance parameters, follow-up time and critical headway were considered a reflection of driver behavior/patterns within Louisiana. The average follow-up time for Louisiana was computed as 3.36 seconds, and the average critical headway was 4.76 seconds. These imply that, in Louisiana, for different drivers using the same circulatory gap to enter a roundabout, the gap between these vehicles is on the average 3.36 seconds. Also, a Louisiana driver stopped at the yield line of a roundabout is most likely to accept a gap of 4.76 seconds or larger.

In comparison to other studies referred to in this report, it is apparent that Louisiana's drivers are less aggressive (maybe due to lack of familiarity) than the average driver in the United States. The national averages of average follow-up time and critical headway from the HCM 6 were 2.60 seconds and 4.70 seconds, respectively. HCM 2010 shows the same parameter as 3.20 seconds and 5.10 seconds, respectively. Other studies recorded critical headways ranging between 2.50 and 5.50 as seen in Figure 28. However, follow-up times in Louisiana are much larger than what is seen from other regions. It is noteworthy that a larger critical headway could suggest caution at roundabouts on the part of drivers departing the yield line to enter the roundabout. This can be associated with driver familiarity, an observation likely to improve as drivers become more familiar with roundabouts [36]. A higher follow-up time could also point to hesitation or lower driver confidence while approaching the roundabout—again, familiarity could enhance performance in this manner. The local capacity model was calibrated, anchored on the value of follow-up times.

Figure 28. Critical headways and mean follow-up times for similar studies



To determine whether geometric features affect the observed follow-up times, inscribed circle diameter (ICD), splitter width, central island diameter, and approach lane width were compared to the follow-up times. Even though follow-up times most correlated with ICD, none of the relationships proved strong enough to be included in the local model calibration.

Other Observations relating to Gap Acceptance Parameters

The research tested the possibility of varying driver behavior across different Louisiana DOTD districts. The qualifying data obtained were only representative of five parishes making up three DOTD districts. No significant difference between both gap acceptance parameters existed across the three districts considered. It is noteworthy that, for follow-up times, the highest was recorded by District 61 (East Baton Rouge and Ascension parishes) and the least was District 3 (Lafayette). For critical headways, the opposite was observed. A possible explanation may point to the traffic volumes and level of congestion common to the two districts: the relatively higher traffic delays within the East Baton Rouge area may cause drivers to be less patient and therefore closely follow each other while entering roundabouts, thus the lower follow-up times. Critical headways are highest in East Baton Rouge, possibly because despite typically moving relatively fast while approaching the roundabouts, the higher volumes in circulatory flow causes drivers to accept higher gaps—when they are absolutely sure it is safe to proceed. The exact opposite is observed for Lafayette—lower traffic volumes and level of congestion allows reasonable travel speeds for drivers in the area, so there is less aggression while approaching the roundabout but critical headways are least in the area because of familiarity to roundabouts. The Lafayette area has the highest number of roundabouts, and therefore local drivers are likely to be more familiar with assessing the risk and accepting smaller gaps. Perhaps, future studies may explore these theories in more detail.

A cluster for functional classes were also created to test the difference in driver behavior across different classes of roads. For local roads, collectors and arterials and gap acceptance parameters showed no differences worth exploring further—drivers showed consistent behavior irrespective of the type of roads they drove on.

Capacity Models

This study used 1,696 bins representing 1,696 minutes to develop and calibrate capacity models. This number, as of the date of this study, is by far the largest ever used in any roundabout capacity calibration effort within the United States. Prior to this feat, the FHWA Report SA-15-070 [4] (based off which the HCM 6 model was developed) recorded the highest number of bins—819 bins.

It was discovered that both the HCM 2010 and HCM 6 roundabout models do not accurately estimate roundabout capacities for Louisiana conditions. For the HCM 2010 model, field-observed data suggests only a slight deviation from the model's predictions. For circulatory flows below approximately 350 pc/hr, the HCM 2010 model slightly overestimated performance, not exceeding 100 pc/hr of approach flows. An underestimation of less than approximately 50 pc/hr occurred for circulatory flows exceeding the same threshold. A similar but worse scenario is observed with the HCM 6 model—the threshold in this case is about 1500 pc/hr of circulatory flows, and overestimation could be as much as approximately 300 pc/hr for lower circulatory flows and an underestimation of only a maximum of approximately 20 pc/hr for higher circulatory flows.

Calibration to local conditions (Model 3) resulted in better estimation than both HCM models. The local model was developed by recalibrating the HCM 6 model to a fixed intercept based on the average observed follow-up time and a slope parameter based on regression of the data. Considering the fitness of the model, the recommended capacity model for Louisiana roundabouts is Equation [18].

$$Cpce = 1072.3e^{-0.0009*Vc} \quad [18]$$

where, the variables remain as previously defined.

The local model results showed that it performed closer to the HCM 2010 model than the HCM 6 model.

Roundabout Performance

Table 19 shows a summary of the field-derived delays and Appendix F gives a detailed report on each roundabout, showing the delays, queue lengths, and LOS. It is evident that for the peak times, most of the assessed approaches perform below expectations, recording a LOS of F. The performance improved better at EF of 1.06 than at EF of 1.2. It would be expected that, existing roundabouts would continue to experience improved performance with driver familiarity. Additionally, new roundabouts designed with the recommended calibrated model will also show better performance than newly developed roundabouts modelled with generic capacity models provided by the HCM.

SIDRA Parameters

The study determined an EF of 1.06 resulted in the lowest RMSE between field-observed capacities and SIDRA-derived capacities. The comparison of field-observed capacities and SIDRA-derived capacities at EF of 1.2 shows SIDRA output underestimating the capacity of each approach. The SIDRA-derived capacities show better match to the field-observed capacities at EF of 1.06. However, the average age of roundabouts assessed was approximately 8 years. Using a Design Build (20 years) EF of 1.0 and extrapolating the EF to a Build Year (zero years) yielded an EF of 1.1.

The study further compared delay, queue length and level of service (LOS) from field to SIDRA estimates at EF of 1.2 and EF of 1.06. However, these parameters were theoretically derived, and not directly measured from field. The ratio of SIDRA to field-derived delay estimates at EF of 1.2 is close to 1 compared to ratio of two estimates at EF of 1.06. A ratio of 1 shows a perfect match. An average delay from all 20 approaches shows field-derived delay of 253.321 sec/veh closely matching to SIDRA output at EF of 1.2 (175.508) compared to EF of 1.06 (87.954). This implies that SIDRA-generated delays at EF of 1.2 were the closest to field-derived delays.

However, the result from comparison of queue lengths derived from field shows closeness to SIDRA-estimated queue lengths at EF of 1.06 compared to EF of 1.2. The ratio of SIDRA-generated queue lengths at EF of 1.06 closely aligns to 1 compared to estimate at EF of 1.2, based on an average field-derived queue length of 42.199 veh closely matching to the average SIDRA queue length of 52.611 veh (at EF of 1.06), compared to 88.788 veh for a EF of 1.2.

Though results on LOS largely depends on the delay (and previous analysis already showing delay at EF of 1.2 closely matching with field-derived delays), a comparison of LOS at each approach from field estimates to SIDRA output shows 87% of approaches matching the LOS at EF of 1.2 compared to 59% at EF of 1.06. It implies SIDRA-generated LOS at EF of 1.2 closely matched the field-derived LOS.

The result shows mixed output with capacities and queue lengths being closer to field derived estimates at EF of 1.06, while the remaining parameters, delay, and LOS show estimates closer to field derived estimates at EF of 1.2. As field delays, queue length, and LOS were theoretically determined rather than directly observed from the field, future studies should find innovative ways to determine these parameters directly from the field to compare with SIDRA outputs. The recently purchased INRIX XD probe data may offer some benefits in exploring an innovative method of collecting such data.

Limitations

The primary limiting condition for this study was the lack of roundabouts with multilane configurations. The research therefore focused on single-lane roundabouts without by-pass lanes, which is the predominant configuration in Louisiana. Also, several existing roundabouts within the state did not experience enough queueing conditions and volumes to be considered for the study, thus limiting the number of roundabouts considered for the study.

It was difficult to capture queue lengths and delays just with the installation of external devices at multiple roundabouts due to which field delays, queue length, and LOS were theoretically determined rather than directly observed from the field. In addition, the research team were unable to retrieve the SIDRA analysis conducted before the roundabouts were built. Thus, the study performed SIDRA analysis at roundabouts that were already familiar to the drivers affecting the estimation of parameters like gap and capacity. This is the reason why the determined EF of 1.06 had to be extrapolated.

Recommendations

Based on the observations and findings from this research, the study recommendations are explained in the following sections. It includes recommendations on the capacity model for single-lane roundabouts, delay and LOS, and queuing.

Capacity Model for Single-Lane Roundabouts

The calibrated approach volume capacity model took a form/structure similar to the HCM models. The model is empirical (exponential regression) and incorporates a gap-acceptance feature as seen in Equation [11].

$$Q = Xe^{(Y)v_{c,pce}} \qquad [11]$$

where,
Q = Approach lane capacity, adjusted for heavy vehicles (pc/h),
X = 3600/$t_{f,]}$,
Y = (t_c-0.5t_f)/3600, and
$V_{c,\,pce}$ = circulating flow rate (pc/h).

Where it is possible to obtain site specific gap acceptance parameters (t_c and t_f) for a project under planning and development, those parameters may be substituted into Equation [11] and used to estimate the capacity of the roundabout.

If such parameters cannot be conveniently obtained for a single-lane roundabout under development, the capacity model shown in Equation [16] should be used.

$$c_{e,pce} = 1072.3e^{(-0.0009)v_{c,pce}} \qquad [16]$$

where,
$C_{e,\,pce}$ = lane capacity, adjusted for heavy vehicles (pc/h), and
$V_{c,\,pce}$ = circulating flow rate (pc/h).

This takes into account the established gap acceptance parameters for Louisiana: a follow-up time of 3.36 seconds and a critical headway of 4.76 seconds obsrved as of 2019 which is higher than the national averages. It was observed that the HCM 6 capacity model overestimated the capacities at Louisiana sites but the HCM 2010 capacity model generated much closer capacities for a given circulatory volume.

Delay and Queue Length

Similar to the HCM's recommendations, Equation [13] is to be used to estimate delays for each approach at single-lane roundabouts in Louisiana. The estimated delays computed from this model should be used together with Table 3 to determine the associated level of service.

$$d = \frac{3600}{C} + 900T\left[x - 1 + \sqrt{(x-1)^2 + \frac{\left(3600/_{C}\right)x}{450T}}\right] + 5 \times \min[x, 1] \qquad [13]$$

where,
d = Average control delay (s/veh),
x = volume-to-capacity ratio of the subject lane,
c = capacity of the subject lane (veh/h), and
T = time period (h) (T = 0.25 for 15-min analysis).

Queue Length

An additional measure of effectiveness to be used for Louisiana roundabouts' performance assessment is the 95^{th} percentile queue. Queue lengths at single-lane roundabouts should be estimated using Equation [15]. Q_{95} is the estimate of the queue length likely to occur during the peak fifteen minutes being analyzed. Where the Q_{95} for an hourly queue is desired, T will equal 1.0 instead of 0.25 as explained below:

$$Q95 = 900T \times \left[x - 1 + \sqrt{(1-\mathrm{x})^2 + \frac{\left(3600/_{C}\right)x}{150T}}\right] \times {}^{\mathrm{c}}/_{3600} \qquad [15]$$

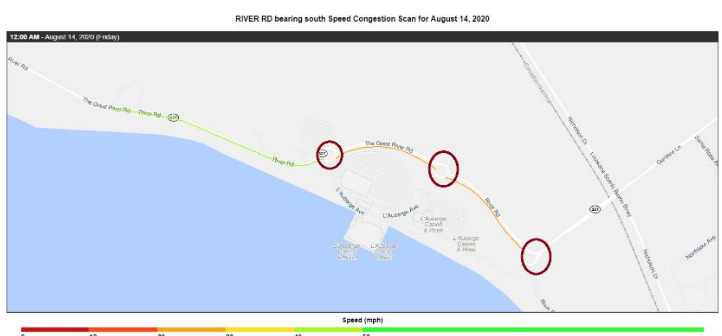
where,
Q_{95} = queue length (veh),
x = volume-to-capacity ratio of the subject lane,
c = capacity of the subject lane (veh/h), and
T = time period (h) (T = 0.25 for 15-min analysis).

SIDRA Environment Factor

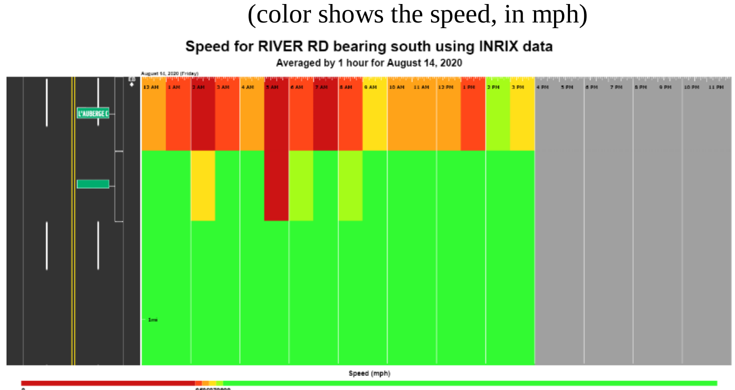
An extrapolated EF of 1.1 is recommended for Build Year when using SIDRA with the HCM 6 capacity model. However, since the study showed mixed output, with capacities and queue lengths closer to field derived estimates at EF of 1.06, while the remaining parameters (delay and LOS) were closer to field derived estimates at EF of 1.2, it may be beneficial for this suggested EF to be validated in the future. More so, field delays, queue lengths and LOS were theoretically determined rather than directly observed from the field, so future studies should find innovative ways to determine these parameters directly from the field to compare with SIDRA outputs. The recently purchased INRIX XD probe data may offer some benefits in exploring an innovative method of collecting these parameters

However, the availability of XD segments and data at all the approaches should be checked carefully before selecting locations for the study. Figure 29 shows excerpts from the congestion scan tab in RITIS platform [37] as a sample to show how queue lengths can be extracted from the data set. In Figure 29 (b), the segment with red color shows the length of queue at certain time of a day. The color largely depends on the selected speed threshold.

Figure 29. Sample XD segments at roundabout approaches [37]



(a) Three roundabouts and their approaches with XD segments (color shows the speed, in mph)



(b) Congestion map showing the segments under congestion.

Recommended Analysis Process

The analysis process to be used for single-lane roundabouts in Louisiana is summarized in Figure 30.

Figure 30. Recommended analysis process for single-lane roundabouts

1.
   - Peak 15-minute flow rate should be determined from field-observed demand volumes
   - The highest 15 minute flow rate within the peak hour is used
2.
   - Adjust flow rates for heavy vehicles
   - $V_{pce}=v/f_{HV}$, $f_{HV}=1/[1+P_T(E_T-1)$, P_T=Heavy Vehicle %, E_T=2.0 for Heavy Vehicles
3.
   - Steps 1 & 2 above should be repeated for circulating and exiting flow rates
4.
   - Compute the capacity of the approach lane in pce using Equation[16]
5.
   - Convert both approach flow rates and approach capacity into Volumes (vehicles per hour) by multiplyinng by f_{HV}
6.
   - Determine the volume to capacity ratio for the approach, x= v/c
7.
   - Calculate the average control delay and corresponding LOS for the approach using Equation[13]
8.
   - Estimate the 95_{th} percentile queue for the approach using Equation[15]

Acronyms, Abbreviations, and Symbols

Term	Description
CID	Central Island Diameter
CSV	Comma Separated Values
DOTD	Department of Transportation and Development
EDSM	Engineering Directives & Standards Manual
GPS	Global Positioning System
HCM	Highway Capacity Manual
ICD	Inscribed Circle Diameter
LA	Louisiana
LOS	Level of Service
LTRC	Louisiana Transportation Research Center
MLE	Maximum Likelihood Estimation
NCHRP	National Cooperative Highway Research Program
NE	North East
NW	North West
pce	Passenger Car Equivalent
pcu	Passenger Car Units
pc/h	Passenger Car per Hour
PRC	Public Review Committee
RER	Roundabout Event Recorder
RMSE	Root Mean Square Error
SD	Standard Deviation
TOPS	Traffic Operations & Safety
UK	United Kingdom
veh	Vehicles
veh/hr	Vehicles per Hour
veh/day	Vehicles per Day

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Appendix A

List of all Roundabout Locations within the State as of 02/28/2019 [38]

SN	District	Intersection	Parish	Letting Date	Selected for this study	Assigned Unique ID for this study
1	2	LA 406 at LA 407	Orleans	9/30/2009	No	
2	2	LA 428 at Mardi Gras	Orleans	9/12/2012	No	
4	3	LA 89 at Chemin Metairie	Lafayette	Local	Yes	1
9	3	LA 92 at Chemin Metairie	Lafayette	Local	Yes	2
10	3	LA 92 at LA 339	Lafayette	5/26/2010	Yes	3
12	3	LA 93 at LA 342	Lafayette	4/4/2003	Yes	4
11	3	LA 93 at LA 3168	Lafayette	7/11/2012	No	
21	3	At Hector Connoly Rd near I-49 Frontage Rd	Cerencro	NA	Yes	6
20	3	LA 93 (Dulles) at Apollo Rd Ext	Scott	NA	Yes	5
8	3	Fortune Rd at Bonin Rd	Lafayette	NA	Yes	7
15	3	LA 342 at LA 724 [Under Construction]	Lafayette	5/10/2017	Yes	17
3	3	LA 726 at Evangeline Court Shopping Center (near I-49 Frontage Rd)	Lafayette	7/13/2016	No	
5	3	LA 89 at LA 92	Lafayette	Local	No	
6	3	LA 92 at Bonin Rd	Lafayette	2/24/2010	No	
7	3	Chemin Metairie Parkway at Viaulet Rd	Lafayette	Not Programmed	No	
13	3	LA 31 at LA 92* [Under Construction]	St. Martin	6/14/2017	No	
14	3	LA 93 at LA 98	Lafayette	7/14/2010	No	
16	3	LA 347 at Doyle Melancon Rd [Under Construction]	St. Martin	4/11/2018	No	
17	3	I-10 Ramps at LA 347 (2) [Under Construction]	St. Martin	9/14/2016	No	
18	3	LA 733 at LA 3095 (Kaliste Saloom Rd) [Under Construction]	Lafayette	Not Programmed	No	
19	3	LA 86 at LA 320 [Under Construction]	Iberia	3/9/2016	No	
22	4	LA 538 at Ravendale [Under Construction]	Caddo	1/13/2016	No	
23	4	US 79/LA 9 bypass	Claiborne	7/1/2016	No	
24	5	Arkansas Road Widening (LA 616 at Good Hope Rd, LA 617, Forty Oaks Farm Rd, & K	Ouachita	11/16/2016	No	
25	7	I-210 and Cove Lane Interchange (south ramp only)	Calcasieu	6/26/2013	No	
26	7	LA 1138-2 (Prien Lake Rd) at Holly Hill (w/other improvements)	Calcasieu	3/11/2015	No	
27	8	LA 184 at LA 468 (Replaces Flashing Beacon)	Vernon	1/10/2018	No	
28	8	US 171 at LA 8/LA 28 *	Vernon	7/21/2010	No	
31	61	LA 327 River Rd at LA 327 Gardere	East Baton Rouge	by Permit	Yes	8
30	61	LA 327 River Rd 3	East Baton Rouge	by Permit	Yes	9
29	61	LA 327 River Rd 2	East Baton Rouge	by Permit	Yes	10
32	61	LA 431 at LA 42	Ascension	11/16/2011	Yes	11
36	62	LA 1091 at Brownswitch	St. Tammany	4/14/2010	Yes	12
38	62	LA 3158 at Old Covington Road	Tangipahoa	8/26/2009	Yes	13
39	62	LA 36 at LA 59 (Abita)	St. Tammany	3/28/2007	Yes	14
46	62	US 190 at LA 434	St. Tammany	11/14/2012	Yes	15

SN	District	Intersection	Parish	Letting Date	Selected for this study	Assigned Unique ID for this study
35	62	LA 1077 at LA 1085	St. Tammany	2/11/2015	Yes	16
33	62	LA 1026 (Lockhart) at LA 1030 (Cockerham)	Livingston	11/1/2015		
34	62	LA 1026 Roundabouts Dunn*	Livingston	10/11/2017		
37	62	LA 16 at LA 22	Livingston	6/11/2014		
40	62	LA 447 at I-12 EB Ramp and WB Ramp (2)** (UNDER CONSTRUCTION)	Livingston	10/1/2015		
41	62	LA 59 at Sharp Road	St. Tammany	12/13/2017		
42	62	Summit Blvd (between US 190 and LA 433)	St. Tammany	3/26/2014		
43	62	US 11 at Cleo Road	St. Tammany	7/13/2016		
44	62	US 190 at Eden Church	Livingston	5/1/2015		
45	62	US 190 at LA 1026 (Juban Rd)*	Livingston	7/12/2017		
47	62	US 51 Business at I-12 EB Ramp, WB Ramp and US 51 Business at Club Deluxe Rd *** (3	Tangipahoa	10/8/2014		

Appendix B

List of Roundabout Locations used for the Study

S. No.	Unique ID	Approaches	DOTD District Name (No)	Intersection	Coordinates	Age, in years	Functional Class
1	1	R3N	3	Chemin Metarie Rd. - LA 89	30.084444, -91.990639		Collector
2		R3S	3	Chemin Metarie Rd. - LA 89		11	Collector
3		R3E	3	Chemin Metarie Rd. - LA 89			Arterial
4		R3W	3	Chemin Metarie Rd. - LA 89			Arterial
5	2	R4N	3	Chemin Metarie Rd. - E. Milton Ave.	30.109222, -92.015611		Collector
6		R4S	3	Chemin Metarie Rd. - E. Milton Ave.		11	Arterial
7		R4E	3	Chemin Metarie Rd. - E. Milton Ave.			Arterial
8		R4W	3	Chemin Metarie Rd. - E. Milton Ave.			Arterial
9	3	R7N	3	Verot School Rd. - E. Milton Ave.	30.109336, -92.032487	9	Arterial
10		R7E	3	Verot School Rd. - E. Milton Ave.			Arterial
11		R7W	3	Verot School Rd. - E. Milton Ave.			Arterial
12	4	R8N	3	Ridge Rd. - Rue du Belier	30.181239, -92.091907	16	Arterial
13		R8S	3	Ridge Rd. - Rue du Belier			Collector
14		R8E	3	Ridge Rd. - Rue du Belier			Arterial
15		R8W	33)	Ridge Rd. - Rue du Belier			Arterial
16	5	R10NE	3	St. Mary St. - Harold Gauthe Rd.	30.243707, -92.106775		Collector
17		R10NW	3	St. Mary St. - Harold Gauthe Rd.		N/A	Arterial
18	6	R11E	3	Hector Connoly Rd. - E. Angelle St.	30.322955, -92.030748	N/A	Collector
19		R11W	3	Hector Connoly Rd. - E. Angelle St.			Collector
20	7	R13S	3	Fortune Rd. - Bonin Rd.	30.123833, -92.007472		Collector
21		R13E	3	Fortune Rd. - Bonin Rd.		N/A	Collector
22	8	R16S	61	Eastern L'Auberge Ave. - River Rd.	30.343299, -91.144287	9	Local
23	9	R17S	61	Gardere Ln. - River Rd.	30.346676, -91.148259	9	Collector
24	10	R18W	61	Western L'Auberge Ave. - River Rd.	30.347302, -91.153282	9	Collector
25	11	R19NW	61	Oak Grove-Port Vincent Hwy - LA 431	30.331530, -90.853300	8	Arterial
26		R19S	61	Oak Grove-Port Vincent Hwy - LA 431			Arterial
27	12	R22E	62	Brownswitch Rd. - Robert Blvd.	30.310361, -89.747778	6	Collector
28		R22W	62	Brownswitch Rd. - Robert Blvd.			Collector
29	13	R23W	62	S. Airport Rd. - Old Covington Hwy.	30.493956, -90.415052	9	Collector
30	14	R24N	62	Level St. - LA 36	30.478287, -90.037674	12	Arterial
31		R24SE	62	Level St. - LA 36			Collector
32		R24E	62	Level St. - LA 36			Local
33		R24W	62	Level St. - LA 36			Arterial
34	15	R25E	62	LA 434 - US 190	30.483330, -90.926538	6	Arterial
35		R25W	62	LA 434 - US 190			Arterial
36	16	R26N	62	Turnpike Rd. - LA 1085	30.46703, -90.18247		Arterial
37		R26S	62	Turnpike Rd. - LA 1085			Arterial
38		R26E	62	Turnpike Rd. - LA 1085		2	Collector
39		R26W	62	Turnpike Rd. - LA 1085			Collector
40	17	R27N	3	Ridge Rd. - LA 342	30.18111, -92.14197	1	Collector
41		R27E	3	Ridge Rd. - LA 342			Collector

Appendix C

Follow-Up Times Computed for Each Site with Qualifying Data

Site No	Roundabout	Approach	No. of Observations	Mean Follow-Up time (s)	SD
1	R3	North	691	3.51	1.34
2	R3	South	617	3.47	1.36
3	R3	East	1981	3.13	1.23
4	R3	West	1924	2.46	1.67
5	R4	North	1157	3.48	1.32
6	R4	South	1096	3.59	1.31
7	R4	West	7092	3.47	1.26
8	R4	East	5886	3.52	1.28
9	R7	East	7993	3.45	1.22
10	R7	North	7064	3.54	1.20
11	R7	West	6713	3.42	1.24
12	R8	East	5589	3.34	1.23
13	R8	North	3888	3.40	1.23
14	R8	South	1006	3.47	1.29
15	R8	West	3766	3.34	1.24
16	R10	NE	1098	3.67	1.24
17	R10	NW	13981	3.40	1.20
18	R13	East	619	3.25	1.31
19	R13	South	2217	3.25	1.31
20	R16	South	2343	3.10	1.13
21	R17	South	144	3.67	1.36
22	R18	West	572	2.70	1.18
23	R19	Northwest	2525	3.35	1.28
24	R19	South	3730	3.36	1.21
25	R22	East	1042	3.62	1.30
26	R22	West	2363	3.48	1.26
27	R23	West	1599	3.47	1.26
28	R24	East	1706	3.76	1.30
29	R24	North	2828	3.71	1.22
30	R24	Southeast	127	3.72	1.50
31	R24	West	7617	3.41	1.09
32	R25	East	11123	2.99	1.13
33	R25	West	3477	3.14	1.08
34	R27	East	1639	3.16	2.3
35	R27	North	345	3.15	1.17

Appendix D

Critical Times Computed for Each Site with Qualifying Data

Site No	Roundabout	Approach	No. of Observations	Mean Critical Headway (s)	SD
1	R3	North	252	4.17	0.64
2	R3	South	155	4.72	1.16
3	R3	East	16	4.46	0.74
4	R3	West	105	5.27	1.79
5	R4	North	466	4.14	0.72
6	R4	South	256	4.74	0.61
7	R4	East	971	4.67	0.92
8	R4	West	241	4.59	0.85
9	R7	East	991	4.37	0.91
10	R7	North	298	3.94	0.63
11	R7	West	24	5.61	1.33
12	R8	East	427	4.99	1.19
13	R8	North	583	4.37	0.98
14	R8	South	491	4.33	0.75
15	R8	West	456	4.47	0.82
16	R10	North East	637	4.80	0.97
17	R10	North West	94	6.11	1.42
18	R13	East	114	4.35	0.83
19	R13	South	29	5.39	2.15
20	R16	South	48	6.10	2.07
21	R17	South	13	6.72	0.03
22	R18	West	21	6.72	2.51
23	R19	Northwest	83	5.01	1.17
24	R19	South	16	5.61	1.33
25	R22	East	219	5.14	0.94
26	R22	West	484	5.29	1.18
27	R23	West	418	5.06	1.30
28	R24	East	514	5.48	1.61
29	R24	North	111	4.84	1.22
30	R24	Southeast	160	4.61	0.81
31	R24	West	77	5.34	1.42
32	R25	East	147	5.58	0.56
33	R25	West	147	6.89	2.70
34	R27	East	7	6.45	0.03
35	R27	North	174	4.58	0.85

Appendix E

Field- and SIDRA-derived Capacities

Site	Approach	Circulating Flow	Observed Capacity	SIDRA-Derived Capacities					
				(EF =1.2)	(EF =1.0)	(EF =1.1)	(EF =1.04)	(EF =1.05)	(EF =1.06)
R3	South	313	1126	1013	1267	1127	1207	1194	1180
		377	1259	977	1229	1091	1171	1157	1144
		645	1160	829	1066	935	1010	997	984
		786	786	742	969	844	916	903	891
		1017	636	585	788	677	741	730	719
R4	North	372	1052	1020	1275	1136	1216	1202	1187
		936	748	669	881	763	831	819	808
		1173	587	430	658	556	615	605	595
		2013	629	247	180	220	195	199	203
		2643	282	247	180	219	195	199	203
R4	South	428	1163	993	1245	1107	1187	1172	1158
		948	822	660	871	755	821	809	798
		778	908	781	1011	884	957	944	931
		692	1008	839	1076	945	1020	1007	994
		498	1058	957	1208	1071	1150	1135	1122
R4	West	260	1235	1078	1332	1193	1273	1259	1245
		1230	984	252	600	492	557	546	536
		1114	852	516	718	615	674	665	654
		754	1196	798	1029	903	975	962	950
		584	1038	907	1152	1017	1095	1081	1068
R7	East	253	1266	978	1228	1091	1170	1156	1143
		1099	775	421	623	523	582	572	562
		822	948	623	837	720	787	776	764
		758	1010	662	883	762	831	819	807
		504	1135	816	1054	921	997	984	971
R7	North	383	1211	890	1135	1000	1078	1064	1051
		909	843	569	773	662	725	714	703
		700	955	698	922	797	869	856	844
		1104	714	416	619	518	577	567	557
		510	1148	812	1050	918	993	980	967
R8	East	129	1227	1031	1275	1141	1218	1205	1192
		775	839	563	772	658	723	711	700
		521	975	717	947	819	892	879	867
		497	1055	732	963	835	909	896	883
		252	1263	913	1156	1022	1099	1086	1072
R8	North	549	915	698	926	801	872	860	847
		1240	522	188	421	288	378	365	352
		966	773	446	638	535	595	585	574
		840	840	524	727	619	679	668	658
		641	898	647	863	741	811	798	786
R8	South	635	889	648	867	745	815	802	790
		1534	191	184	149	170	159	162	164
		1267	571	188	396	213	348	334	318
		1029	708	403	592	493	551	540	531
		903	516	486	682	573	635	624	614
R8	West	189	1449	970	1215	1081	1159	1145	1131
		837	773	528	729	618	682	670	659
		752	940	576	787	674	738	727	715
		563	1064	690	917	792	864	851	838
		441	1198	768	1004	877	949	936	923

Site	Approach	Circulating Flow	Observed Capacity	SIDRA-Derived Capacities					
				(EF =1.2)	(EF =1.0)	(EF =1.1)	(EF =1.04)	(EF =1.05)	(EF =1.06)
R16	South	64	1414	1240	1511	1364	1449	1434	1418
		711	776	717	964	826	905	891	878
		511	766	846	1110	965	1048	1034	1020
		457	1305	885	1153	1006	1090	1075	1061
		192	1345	1101	1375	1225	1311	1296	1281
R19	Northwest	1	1243	1165	1398	1271	1344	1332	1320
		743	172	714	930	811	878	864	856
		717	314	729	949	826	897	885	872
		435	870	887	1124	994	1069	1056	1043
		187	1121	1030	1271	1139	1216	1203	1189
R19	South	186	990	1030	1271	1139	1215	1201	1188
		947	568	586	787	677	740	729	718
		653	832	768	993	868	940	928	915
		395	1249	911	1148	1017	1092	1079	1067
		320	1085	951	1191	1060	1135	1122	1109
R22	East	454	843	775	1001	877	948	936	923
		998	561	448	637	536	594	584	574
		747	690	607	813	699	764	752	741
		651	716	661	875	757	824	813	801
		611	734	685	902	782	851	838	827
R22	West	391	848	815	1044	918	991	978	966
		914	588	509	698	593	653	643	632
		816	816	571	766	656	719	708	698
		643	837	666	880	762	830	818	806
		549	853	720	940	819	889	877	864
R24	East	130	782	1052	1297	1164	1239	1226	1212
		852	524	582	789	674	740	729	717
		584	778	741	971	844	916	904	891
		485	849	800	1037	906	982	968	955
		189	1071	1004	1247	1114	1191	1177	1164
R24	North	795	795	620	830	713	780	768	757
		387	839	861	1101	970	1045	1032	1019
		382	1018	865	1105	973	1049	1036	1023
		304	1098	917	1159	1027	1104	1090	1076
		368	1043	874	1114	982	1058	1045	1032
R24	South	743	434	648	865	744	813	801	789
		888	508	218	764	651	465	453	443
		1204	301	373	507	391	545	534	525
		1114	310	469	587	485	618	608	598
		1021	389	560	663	558	715	705	693
R24	West	127	1393	1056	1299	1167	1243	1229	1217
		513	1027	784	1018	889	963	950	937
		450	1028	822	1060	929	1004	990	978
		325	1236	905	1146	1012	1089	1076	1063
		252	1137	955	1198	1065	1142	1128	1115
R25	East	126	1455	1171	1428	1288	1369	1355	1340
		713	713	844	1084	952	1027	1014	1001
		480	1021	990	1247	1105	1186	1172	1159
		325	1172	1072	1332	1189	1271	1256	1243
		183	1339	1142	1401	1260	1340	1326	1313

Appendix F

Field- and SIDRA-derived Delay, LOS, and Queue Length

Site	Approach	Field Derived			SIDRA Estimates					
		Delay, in sec/veh	Level of Service (LOS)	Queue length, 95th, in veh	Delay at EF of 1.2, sec/veh	LOS at EF of 1.2	Queue length at EF of 1.2, veh	Delay at EF of 1.06, sec/veh	LOS at EF of 1.06	Queue length at EF of 1.06, veh
R3	South	200.878	F	47.158	82.800	F	75.600	35.200	E	41.400
R3		313.500	F	66.948	153.400	F	119.000	76.600	F	75.800
R3		443.498	F	73.683	203.100	F	125.500	108.500	F	81.900
R3		250.845	F	38.647	72.900	F	40.400	29.900	D	19.200
R3		254.167	F	32.258	89.100	F	37.700	35.100	E	17.400
R4	North	193.142	F	43.389	56.400	F	52.300	24.900	C	25.900
R4		311.502	F	41.322	95.400	F	46.700	38.700	E	22.200
R4		297.790	F	32.528	204.800	F	65.900	59.800	F	25.400
R4		1220.490	F	58.958	736.200	F	142.000	989.100	F	162.300
R4		923.428	F	26.098	143.400	F	26.600	246.900	F	41.100
R4	South	290.495	F	59.701	105.400	F	84.300	46.300	F	48.000
R4		390.213	F	50.268	143.700	F	69.900	62.500	F	36.900
R4		345.487	F	51.948	107.300	F	62.500	44.600	E	31.400
R4		364.409	F	58.766	121.100	F	76.300	52.600	F	40.900
R4		269.986	F	52.595	81.500	F	62.800	34.100	D	32.100
R4	West	227.414	F	55.062	94.200	F	92.100	42.000	E	55.600
R4		830.758	F	80.834	1348.800	F	261.700	402.000	F	160.200
R4		555.836	F	60.755	322.000	F	123.000	167.100	F	81.500
R4		563.369	F	84.291	246.800	F	144.500	141.500	F	100.100
R4		311.976	F	55.808	97.200	F	68.200	40.800	E	35.400
R7	East	239.094	F	57.960	155.500	F	129.800	79.000	F	86.900
R7		457.156	F	51.170	409.100	F	129.300	203.300	F	86.100
R7		411.066	F	58.816	261.100	F	121.100	138.600	F	80.300
R7		414.564	F	62.658	261.600	F	129.000	141.700	F	86.900
R7		323.191	F	61.755	199.900	F	125.000	105.100	F	82.100
R7	North	289.760	F	61.898	185.400	F	130.300	97.000	F	86.500
R7		381.589	F	50.910	245.500	F	104.300	124.100	F	66.100
R7		329.361	F	53.134	192.900	F	101.600	94.200	F	62.000
R7		393.571	F	44.333	355.000	F	111.200	163.000	F	68.700
R7		335.694	F	63.653	209.200	F	129.800	112.000	F	86.500
R8	East	152.802	F	43.345	112.000	F	124.000	52.500	F	80.500
R8		287.062	F	43.981	249.700	F	106.200	124.200	F	67.200
R8		230.874	F	44.822	188.700	F	105.400	90.900	F	65.300
R8		267.461	F	52.192	223.700	F	126.500	116.900	F	83.800
R8		236.974	F	57.541	195.000	F	149.000	106.400	F	104.700
R8	North	208.099	F	39.979	168.700	F	92.000	76.000	F	53.900
R8		262.817	F	27.604	849.600	F	128.500	259.300	F	71.000
R8		355.728	F	45.450	360.300	F	120.900	188.400	F	82.200
R8		330.506	F	47.304	300.200	F	118.200	156.300	F	78.700
R8		249.483	F	43.422	203.200	F	100.500	99.500	F	62.400
R8	South	239.527	F	42.094	196.500	F	97.600	93.100	F	59.100
R8		44.904	E	4.837	127.400	F	17.300	176.900	F	23.800
R8		340.276	F	33.834	965.800	F	145.800	398.300	F	98.600
R8		334.278	F	40.709	374.800	F	114.000	185.600	F	75.000
R8		96.389	F	16.366	87.200	F	31.400	33.400	D	14.400
R8	West	290.441	F	73.271	241.500	F	197.200	147.600	F	149.900
R8		270.932	F	39.642	240.100	F	95.700	115.500	F	58.700
R8		353.428	F	54.247	310.700	F	134.400	169.800	F	92.900

Site	Approach	Field Derived			SIDRA Estimates					
		Delay, in sec/veh	Level of Service (LOS)	Queue length, 95th, in veh	Delay at EF of 1.2, sec/veh	LOS at EF of 1.2	Queue length at EF of 1.2, veh	Delay at EF of 1.06, sec/veh	LOS at EF of 1.06	Queue length at EF of 1.06, veh
R8		315.814	F	57.465	268.600	F	140.700	147.900	F	97.200
R8		320.238	F	64.635	273.800	F	161.600	158.100	F	116.200
R16		199.320	F	57.678	89.100	F	152.300	40.400	E	89.200
R16		200.524	F	33.887	81.700	F	46.300	30.400	D	21.300
R16	South	100.588	F	22.952	34.100	D	24.800	17.100	C	13.100
R16		397.634	F	78.199	234.400	F	159.500	127.800	F	108.800
R16		241.868	F	61.685	123.700	F	127.400	57.100	F	82.000
R19		100.253	F	34.103	64.600	F	474.100	30.800	D	37.700
R19		11.360	B	1.329	7.800	A	1.100	6.200	A	1.000
R19	North	17.395	C	3.407	10.700	B	2.600	8.200	A	1.900
R19		124.356	F	28.659	47.200	E	36.500	22.100	C	18.400
R19		133.993	F	37.076	74.000	F	81.700	32.900	D	44.700
R19		78.986	F	24.677	39.300	E	47.700	20.000	C	21.600
R19		154.210	F	22.592	56.000	F	22.500	25.100	D	11.400
R19	South	209.388	F	36.881	80.000	F	47.100	33.000	D	23.000
R19		319.757	F	67.149	189.500	F	134.000	104.100	F	91.300
R19		183.060	F	43.241	94.800	F	79.100	41.300	E	45.600
R22		118.180	F	27.133	80.800	F	52.700	33.300	D	27.000
R22		172.392	F	23.698	157.600	F	52.500	58.800	F	24.200
R22	East	155.654	F	26.707	104.600	F	48.000	41.600	E	23.100
R22		129.421	F	24.921	83.900	F	42.900	33.800	D	20.700
R22		123.316	F	24.776	79.400	F	42.600	32.300	D	20.700
R22		95.088	F	24.161	65.000	F	47.500	27.800	D	24.200
R22		155.582	F	23.358	117.300	F	44.100	45.800	E	20.600
R22	West	294.178	F	43.432	224.000	F	95.500	10.600	F	6.400
R22		207.400	F	36.883	147.900	F	75.300	64.500	F	41.500
R22		168.461	F	33.416	117.800	F	66.800	49.100	E	35.700
R24		22.919	C	9.406	16.300	C	8.800	11.400	B	5.400
R24		84.664	F	15.474	43.000	E	17.100	21.000	C	9.500
R24	East	138.426	F	27.706	70.100	F	41.700	28.700	D	20.200
R24		135.248	F	29.432	71.300	F	48.000	29.300	D	23.800
R24		112.928	F	32.278	67.900	F	75.600	29.700	D	39.700
R24		263.598	F	40.075	12.400	F	7.500	9.500	F	5.500
R24		89.422	F	23.154	46.400	E	37.100	21.600	C	18.800
R24	North	179.677	F	40.436	110.600	F	80.400	47.100	E	46.000
R24		181.364	F	43.465	117.100	F	93.300	52.100	F	56.700
R24		186.037	F	42.144	117.000	F	85.800	50.800	F	50.200
R24		31.499	D	7.386	19.400	C	7.200	12.700	B	4.900
R24		86.294	F	15.255	45.300	E	17.200	21.700	C	9.500
R24	South	48.721	E	7.368	238.000	F	41.000	27.000	D	7.400
R24		40.600	E	6.705	46.900	E	11.100	19.200	C	5.600
R24		55.759	F	9.777	39.200	E	11.600	19.700	C	6.900
R24		225.952	F	61.255	164.800	F	167.300	91.500	F	121.400
R24		259.441	F	50.089	166.400	F	101.200	78.900	F	61.300
R24	West	222.772	F	46.083	140.800	F	92.300	63.500	F	54.400
R24		267.077	F	60.328	187.400	F	136.400	101.000	F	93.300
R24		173.758	F	43.726	113.900	F	98.600	51.400	F	61.500
R25		253.621	F	68.166	130.800	F	151.300	67.900	F	104.200
R25		155.870	F	27.475	26.900	D	15.300	7.700	A	2.400
R25	East	235.900	F	47.285	57.300	F	47.600	24.900	C	23.300
R25		232.297	F	53.133	75.100	F	72.100	32.100	D	38.400
R25		233.635	F	60.223	103.100	F	113.400	48.100	F	73.400

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