2014

THE EFFECT OF ADDITIONAL LANE LENGTH ON ROUNDABOUT DELAY

Samuel Hammond
University of Rhode Island, samham@my.uri.edu

THE EFFECT OF ADDITIONAL LANE LENGTH ON ROUNDABOUT DELAY

BY

SAMUEL HAMMOND

A DISSERTATION SUBMITTED IN PARTIAL FULFILLMENT OF THE

REQUIREMENTS FOR THE DEGREE OF

DOCTOR OF PHILOSOPHY

IN

CIVIL AND ENVIRONMENTAL ENGINEERING

UNIVERSITY OF RHODE ISLAND

2014

DOCTOR OF PHILOSOPHY DISSERTATION

OF

SAMUEL HAMMOND

APPROVED:

Dissertation Committee:

Major Professor: Christopher Hunter

Leon Thiem

Farhad Atash

Nasser H. Zawia
DEAN OF THE GRADUATE SCHOOL

UNIVERSITY OF RHODE ISLAND
2014

ABSTRACT

Currently, there is no United States guideline on how the additional lane lengths affect roundabout operation. The purpose of this research is to provide an insight on how the use of an additional lane as an approach affects roundabouts. Hence, most transportation professionals refer to studies conducted overseas that do not necessarily translate directly to domestic roundabout design and operation. As interest continues to grow in the deployment of modern roundabouts in the United States, there is the need to provide effective information to professionals on roundabout design and its effect on operations. Because of this, the purpose of this research is to provide insight on how the use of an additional lane on an approach affects roundabout operations.

Using delay as the measure of effectiveness, a hypothetical four-leg, double-lane roundabout with additional lane design at both entry and exit is analyzed. The additional lane lengths are varied at both entry and exit in order to study the effect of different additional lane lengths on roundabout operation. Similar length variations are applied to an existing roundabout with known data after calibration and validation. The research indicated that very long additional lane lengths resulted in higher speeds on the approach, but were not necessarily providing the greatest overall impact in reducing delay through the roundabout. Through the analyses of both hypothetical and existing roundabout models, there are diminishing returns on reduction of overall delay as the additional length increases or there are distinct distances where one sees less change per additional increase in the approach length. This research indicated that approximately 150 feet is

that distinct length. Varying the lengths was also found to be more effective when applied to all legs at the same time with the exits.

Findings from this study are intended to provide transportation professionals quantitative means of improving existing roundabout operational performance and also help design future roundabouts with appropriate additional lane lengths that yield better performance. While the design of an additional lane differs from a flared entry, findings from this study can also be applied to flare lengths if they are designed to operate in a similar fashion as additional lane entry.

ACKNOWLEDGEMENTS

First of all, I want to show my great appreciation to my respected advisor, Dr. Christopher Hunter for his support and guidance during my dissertation. In the trials and tribulations of this process, it would not have been possible without the support, guidance, encouragement, and confidence in me from my advisor. He is a patient, knowledgeable, experienced and distinguished professor. It is my luck and great honor to be his student.

Also, I am very thankful to Dr. K. Wayne Lee, Dr. Farhad Atash, Dr. Leon Thiem, Dr. Natacha Thomas and Dr. Peter August for serving on my advisory committee. I am very grateful to Dr. Thomas for her help and constructive suggestions on my dissertation through several meetings that improves its quality greatly.

Many thanks are due to Dr. Kevin Chang of the University of Idaho, Lee Rodegerdts of Kittelson & Associates, Inc., and Richard Tetreault of Vermont Agency of Transportation. I am very grateful to Dr. Chang for his contribution to the operations and statistical aspect of this dissertation. Mr. Rodegerdts provided the field data used in the calibration and validation of the model used in this research. Mr. Tetreault provided the geometric parameters for the roundabout used in this research.

Finally, I'm truly indebted to my family for their love, support, and encouragement.

TABLE OF CONTENT

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1 : INTRODUCTION

1.1 Background Information

Modern Roundabouts as we know them today started as traffic circles in the US in 1905. In the earlier version, the operation of the traffic circle was such that circulating traffic would yield to entering traffic. This configuration resulted in high crash rates and substantial traffic delays. Many were eliminated and found to be undesirable as of the mid 1900s. In the early 1960s, British engineers modified the configuration of traffic circle to yield lower speed, crash rate and delay. They introduced the "give-way" rule, which required entering traffic to yield to circulating traffic. This rule proved to be a much more efficient intersection than the traffic circles, and in many cases, signalized intersections.

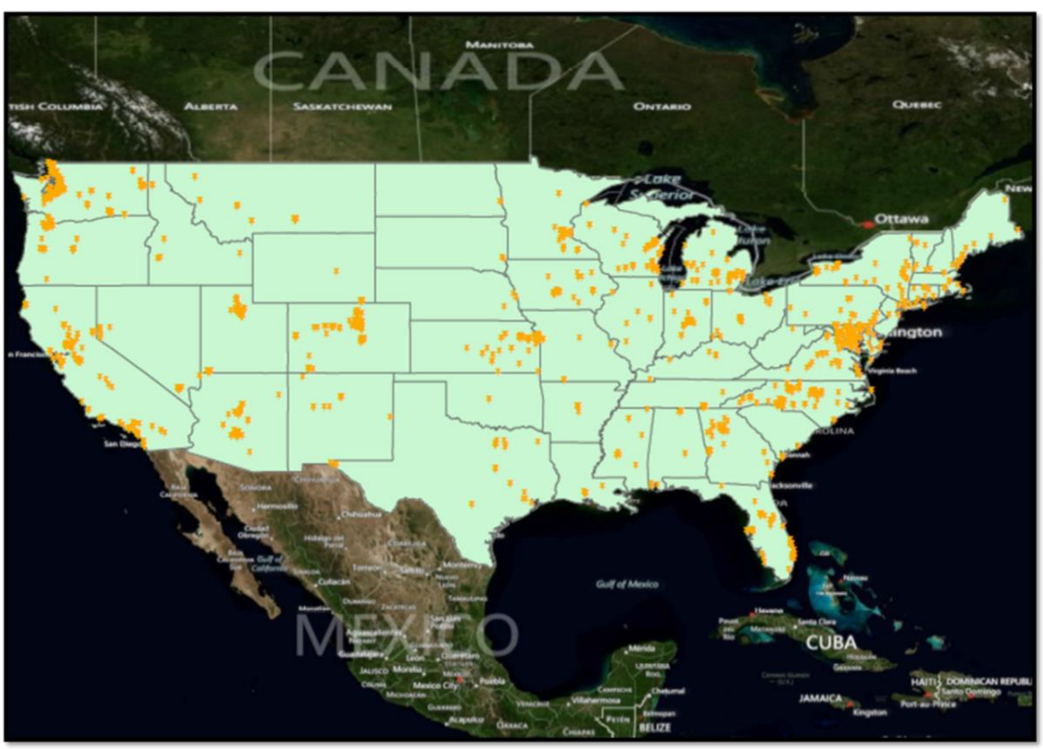

Figure 1:1 Roundabouts in the U.S.A. as of 2012

The traffic circle was reintroduced in the U.S. in the 1990s as a modern roundabout with a new design configuration and operating rules. The first modern roundabout was built in Nevada in 1990. Since then, there has been dramatic growth, and as of December 2012, more than two thousand have been constructed (Kittelson 2012). Figure 1:1 shows the approximate locations of all roundabouts in the U.S. as of December 2012. The most popular two basic roundabout types in the U.S. are: single lane and multi-lane. Single-lane roundabouts have single-lane entries at all approaches and one circulating lane. Multilane roundabouts have at least one entry or exit with two or more lanes and more than one circulating lane. Figure 1:2 shows an example of a multilane roundabout in Springfield, Oregon. In this case the roundabout has two circulatory and entry lanes and can also be classified as a double lane roundabout.

Figure 1:2 Double Lane Roundabout in Springfield, OR

1.2 Problem Definition

As roundabouts have become increasingly popular in the United States, it is very important to establish some means of improving their performance in the near future when vehicle demand nears or exceeds capacity. At signalized intersections, U.S. transportation professionals regularly consider numerous parameters such as green time, cycle length and number of lanes to adjust in order to improve traffic operation. However, there has not been much research performed domestically that addresses how to vary different geometric parameters to improve operations for a roundabout when analysis shows that a nearby development will impact traffic operation. Hence, most transportation professionals refer to studies conducted overseas that do not necessarily translate directly to U.S. roundabout design and operation.

One of the design requirements that needs further exploration is the entry approach. The entry can be designed to increase capacity by either adding a full lane upstream of the roundabout or by widening the approach gradually (flaring) through the entry geometry (NCHRP 2010). Most of the studies on roundabout entry design have been looking at the widening effect of the width of the approach lane. However, little attention has been given to the length of the approach over which to widen the lane and its effect on roundabout operation. Research indicates that no guidance exists to determine when flaring or adding full lane upstream of the roundabout is feasible (justifiable), i.e., when one should be included to improve roundabout operations and safety. And where feasible, there is no guidance as to how long the length should be to improve operations and safety. One key question must be undertaken when assessing whether flaring or adding a full lane upstream should be included in roundabout design. That is how do we identify and quantify the length of the flared area or additional full lane that can improve operational parameters such travel time and delay. Figures 1:3 and 1:4 indicate differences between flaring and additional lane length.

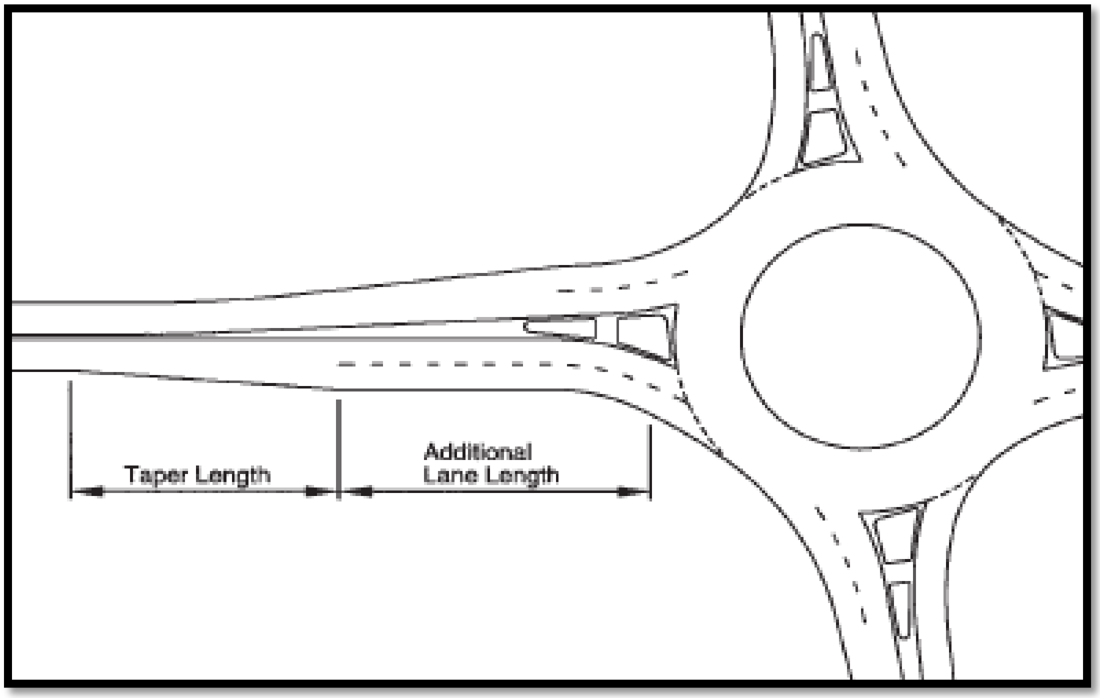

Figure 1:3 Approach Widening by Adding Full Lane

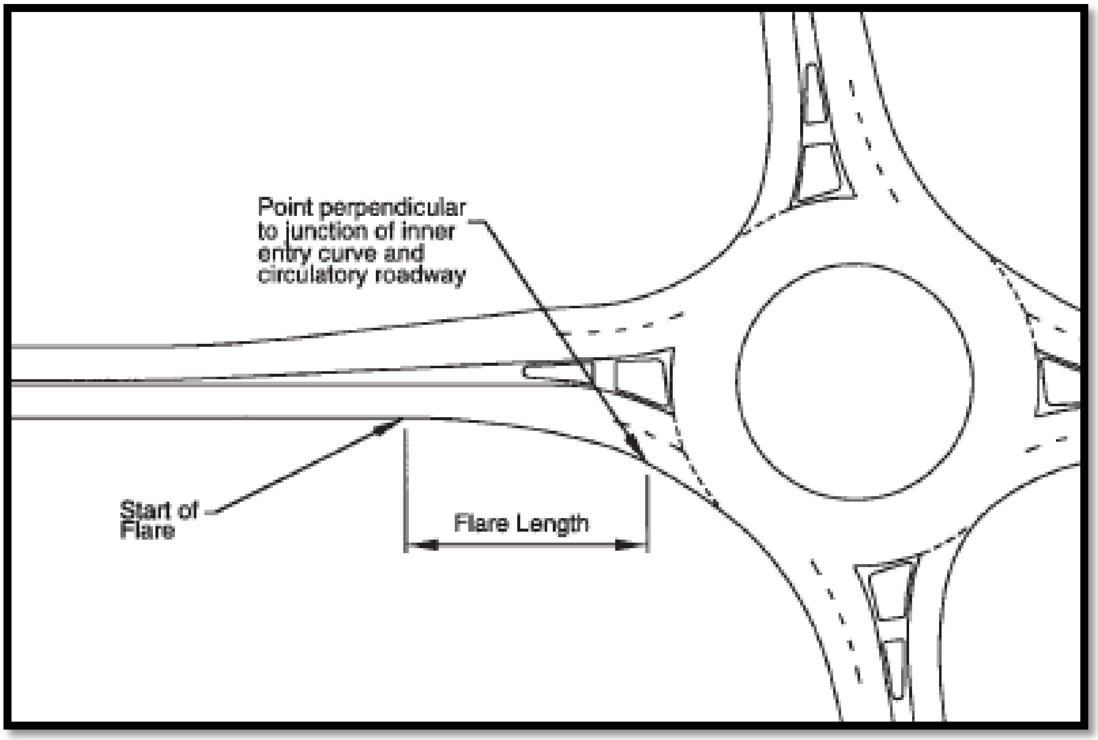

Figure 1:4 Approach Widening by Entry Flaring

In general, the increasing popularity of roundabouts in the U.S. underscores the need for more research on roundabouts in the United States to address issues that traffic engineers face in practice. The means of improving signalized intersections to meet specified demands has been well researched and documented; methods to predict their performances are well established. However, roundabouts lack such research on performance improvement. Thus, this research is conducted to determine the value of flare or additional lane lengths to quantify their contribution to overall roundabout operational performance in terms of a design guideline for approaches.

1.3 Research Objectives and Contributions

The ultimate objective of this research is to have a better understanding of a double-lane roundabout flare or additional lane design and the effectiveness of various lengths in reducing delay at a double-lane roundabout. Towards this end, this dissertation aims and focuses on a double-lane roundabout with additional lane at the entry and the findings applied to roundabout with flare design as well.

Specific objectives are to:

1. Present a framework for examining the effect of additional lane length on a double-lane roundabout operation.

2. Determine the operational impact of additional lane length in roundabouts and quantify the reduction of delay and travel time.

3. Provide transportation professionals with a means of improving existing roundabout operational performance, which should aid during the planning and design stages so that future roundabouts can be built with appropriate additional lane lengths to yield better performance

This dissertation aims to articulate a more thorough understanding of double-lane roundabout additional lane design characteristics and performance, by identifying and quantifying the effect of different lengths on delay. Quantifying and assessing the impact of flare or additional lane length on roundabout operation can be helpful to practitioners who are considering the use of adding a full lane in a roundabout design and to have a better understanding of the effectiveness of various lengths to roundabouts performance.

1.4 Scope

In this dissertation, interest focuses on double-lane roundabouts with a one lane approach that increases to a two lane at the entry. Different entry lane lengths combinations are considered in conjunction with different exit lane lengths. Entry and exit lane lengths less or equal to 150 feet were considered to be short lengths and lengths greater than 150 feet were considered to be longer lengths.

For the research effort, two roundabout situations are modeled extensively. One is a hypothetical four-leg, double-lane roundabout with additional lane design at both entry and exit approaches. The additional lane lengths are varied at both entry and exit in order to study the effect of different additional lane lengths on roundabout operation. A situation of no additional length (or just one lane) is used as well for base comparison. Similar length variation analyses are applied to an existing roundabout with known data after calibration and validation. The results from the analyses of both models are studied to understand the effect that additional lane lengths have on roundabout operations. Delay is the measure of effectiveness used in this study.

1.5 Dissertation Organization

This dissertation is organized in ten chapters:

- Chapter 1 introduces background, objectives, and the scope of the research.

- Chapter 2 presents a literature review of existing information on roundabouts and entry lane design related to operation. Additional literature relevant to the available roundabout guidelines in the U.S is presented. This chapter also presents a review of the microsimulation model used in this study.
- Chapter 3 summarizes how the research effort was conducted with respect to modeling the impact of shared short lane length on roundabout operation and discusses the research hypothesis.
- Chapter 4 describes the models used in this research, their developments and the different scenarios used in this study. It also presents macroscopic analysis of the model using the HCM, the calibration and validation procedure, travel time and delay analysis of the model using VISSIM.
- Chapter 5 includes the results and discussion of the roundabout operational performance and summarizes overall findings.
- Chapter 6 presents conclusions, recommendations on additional lane length design
- Chapter 7 discusses the direction for future work pertaining to additional lane effect on roundabout delay.
- Chapter 8 presents the literature cited in this dissertation as references.
- Chapter 9 presents appendices including supporting documentation on data, analysis methodologies, simulation and modeling outputs. The appendices are:

  • Appendix A: presents key field data from NCHRP 572 report used in this study

• Appendix B: presents initial VISSIM simulation data for hypothetical and existing models before calibration

• Appendix C: presents the calibration data for the different trials

• Appendix D: presents T distribution table with selected critical values highlighted and passenger car equivalent for heavy vehicles table

- Chapter 10 presents list of sources (books, journals etc) which were used to perform this research but not actually quoted in this research.

CHAPTER 2 : LITERATURE REVIEW

Due to the technical nature of this study, it is necessary that basic concepts dealing with roundabouts be defined. In this chapter, first, a description of the basic features of roundabout is presented. This is then followed by the operational performance of roundabouts with emphasis on the key parameters that affect performance. The next section takes a closer look at the entry capacity of roundabout. The following sections present a description of traffic simulation car-following model and its application in the widely-used microscopic traffic simulation model VISSIM. The last section summarizes the findings from the literature review.

2.1 Modern Roundabout

The term modern roundabout and roundabout are used interchangeably throughout this dissertation. A roundabout is a form of circular intersection with a yield control at the entry and appropriate geometric curvature to slow vehicles through the intersection. The term "modern roundabout" is used in the United States to differentiate roundabouts from the older and often large diameter non-conforming traffic circles, rotaries or very small traffic calming circles used on residential streets. Roundabouts as we know today evolved out of traffic circles where circulating vehicles had to yield to entering vehicles. Traffic circles fell out of favor in the U.S. by the mid 1950's because they encountered safety and operational problems as traffic volumes increased beyond their operational thresholds. However, roundabout design was revised in the U.K. where they introduced the yield at entry and the geometric features to reduce vehicle speed. The revised design solved the problems of the existing rotaries and traffic circles. Figure 2:1 illustrates key geometric elements and Table 2:1 describes the key geometric elements of a modern roundabout.

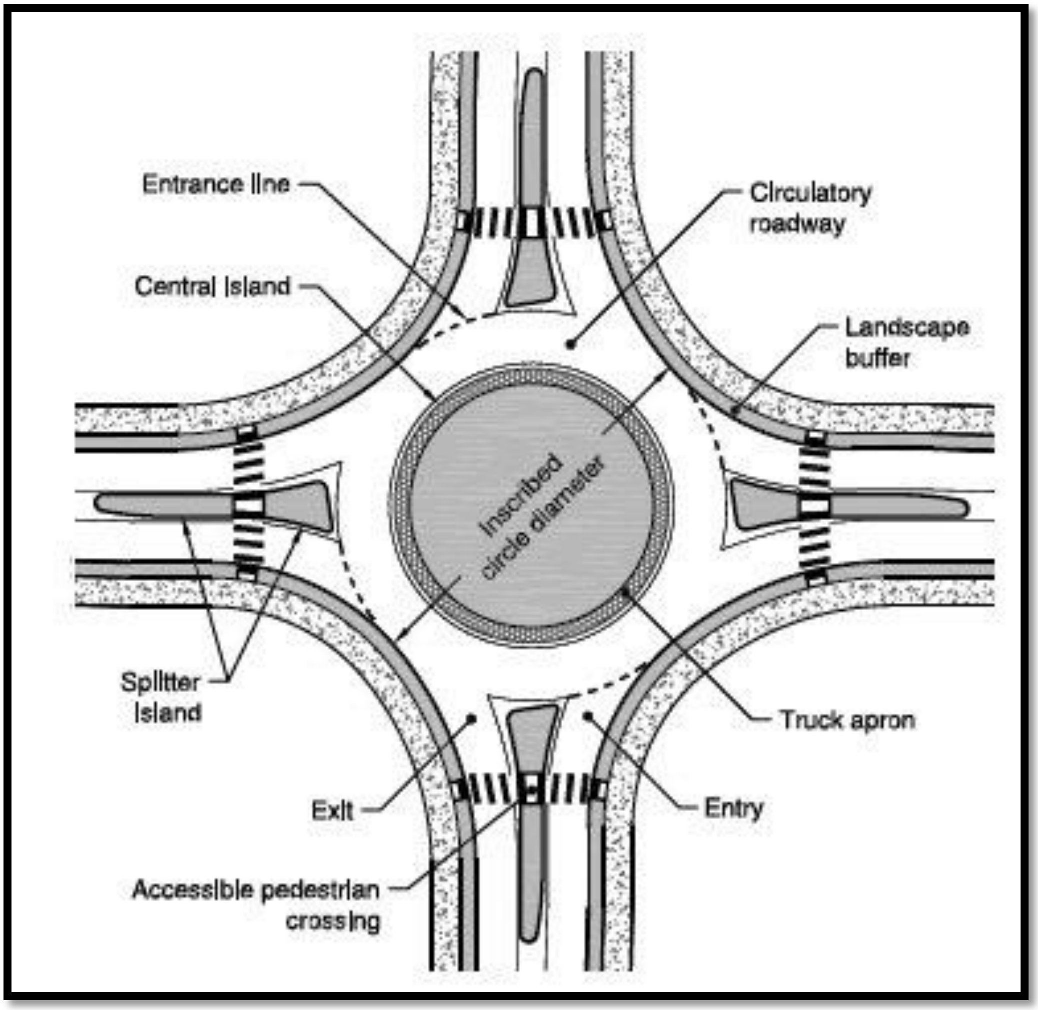

Figure 2:1 Modern Roundabout Geometric Features

Element	Description
Inscribed Circle Diameter	A diameter may range between 50 feet and 300 feet for the circular section.
Circulating Roadway Width	The curved path used by vehicles to travel in a counterclockwise fashion around the central island. The width of the circulatory roadway depends mainly on the number of entry lanes and the radius of vehicle paths.
Central Island	A raised curb usually delineates the central island, and the width of the circulatory roadway and the diameter of the inscribed circle determine its size. Usually, this island is landscaped.
Truck Apron	The apron is usually designed as a mountable portion of the central island to accommodate the wheel path of oversized vehicles.
Splitter Island	A splitter island is placed within the leg of a roundabout to separate entering and exiting traffic.
Bypass Lane	A slip lane is a right lane provided adjacent to the roundabout circular lanes that allows heavy right-turning movements to bypass the roundabouts**.**
Crosswalk	The pedestrian access is limited to crossing the roundabout approaches behind the yield line.
Approach Width	The approach width is the half of the roadway that is approaching the roundabout.
Departure Width	The departure width is the half of the roadway that is departing the roundabout.
Entry Width	The entry width is the perpendicular distance from the right curb line of the entry to the intersection of the left edge line and the inscribed circle.
Exit Width	The exit width is the perpendicular distance from the right curb line of the exit to the intersection of the left edge line and the inscribed circle.
Flare	A flare may be used to increase the capacity of a roundabout by providing additional lanes at the entry.
Entry Angle	To provide the optimum deflection for entering vehicles, the angle of entry should be approximately 30 degrees.
Entry Radius	The entry radius is the minimum radius of curvature measured along the right curb at entry.
Exit Radius	The exit radius is the minimum radius of curvature measured along the right curb at an exit

Table 2:1 Roundabout Elements Description

The modern roundabout is defined by three basic principles:

1. Yield- at-Entry - Vehicles approaching the roundabout must wait for a gap in the circulating flow, yield, before entering the circle.
2. Deflection - Traffic entering the roundabout is directed or channeled to the right with a curved entry path into the circulating roadway.
3. Geometric Curvature - The radius of the circular road and the angles of entry are designed to slow the speed of vehicles.

Using the principle that entering traffic yields to circulating traffic, roundabouts proved to be a much more efficient intersection than the rotaries, and in many cases, signalized intersections. Requiring entering traffic to yield circulating traffic prevents the intersection from locking up. Adequate horizontal curvature of entering and exiting vehicle paths reduces the entry and circulating speeds, which improves safety by reducing the severity of crashes.

Based on the NCHRP Report 672 and the 2000 FHWA Roundabout Informational Guide, roundabouts can be classified into six types with differing applications:

1. Mini-roundabouts.
2. Urban compact roundabouts.
3. Urban single-lane roundabouts.
4. Urban double-lane roundabouts.
5. Rural single-lane roundabouts.
6. Rural double-lane roundabouts.

Roundabouts have been reclassified into three basic categories based on size and number of lanes:

1. Mini-roundabouts
2. Single-lane roundabouts
3. Multilane roundabouts

The three main roundabout categories can be further subdivided by their location such as rural, urban, and suburban. For a roundabout in an urban environment, the inscribed circle diameter tends to be smaller due to smaller design vehicles and existing right-of-way restrictions. The mini-roundabouts are small single-lane roundabouts generally used in low-speed urban environments, with average operating speeds of 35mph or less. Mini-roundabouts are typically useful in low-speed urban environments where conventional roundabout design not feasible due to limited right of way. Single-lane roundabouts have single-lane entries at all legs and one circulating lane. A single-lane roundabout has a bigger inscribed circle diameter than a mini-roundabout. Single lane roundabouts typically have mountable raised splitter islands, a mountable truck apron, and a central island, which is typically landscaped. The multilane roundabouts have at least one entry or exit with two or more lanes and more than one circulating lane.

2.2 U.S. Roundabout Guidelines

This section summarizes existing roundabout guides in the U.S and how they aided the research effort. Currently there are three guidelines on roundabout in the US namely NCHRP 572 - Roundabouts in the United States and NCHRP 672, Roundabouts: An Informational Guide, Second Edition. The NCHRP Report 572 is based on a study of

31 roundabout operations for US conditions. Entry flow, conflicting flow, exit flow, average delay and queue data were collected at these 31 sites and used in the preparation NCHRP Report 572. The data captured at one of the sites in Vermont was used in this research for calibration and validation purposes.

The operational findings and recommendations from NCHRP Report 572 form the basi s of the procedures outlined in the 2010 Highway Capacity Manual. The roundabout model used in 2010 HCM is a macroscopic model. It is based on studies on the 31 sites in the US. The 2010 HCM used findings from studies of these sites to develop the macroscopic model for analyzing roundabout operation. The model is a combination of simple, lane-based regression and gap-acceptance models.

The NCHRP Report 572 proposed exponential regression models of capacity for singlelane and two-lane roundabouts. This report also provides the operational performance model that is recommended for the entry capacity at single-lane roundabouts as shown in Equation 3.2. In addition this report provides the geometric design findings on pedestrian and bicyclists behavior at roundabouts.

NCHRP Report 572 also confirms that roundabout geometry alone is not sufficient for modeling capacity of roundabouts, and driver behavior parameters are the most important parameters affecting roundabout performance. Recently, NCHRP report 672 updates the first edition of the Roundabouts: An Informational Guide, (FHWA 2000). It incorporates some findings from the NCHRP Report 572 and some insights on HCM

2010. It includes roundabout considerations, planning, operational analysis, safety, geometric design, implementing traffic control devices at roundabouts, illumination, landscaping and construction and maintenance. Geometric guidelines from the NCHRP Report 672 were used in setting up the hypothetical model used in this research.

2.3 Operational Performance

Earlier research on roundabout operation was started by the U.K. based Transport and Road Research Laboratory (TRRL), where numerous experiments and observations were performed on existing roundabouts. Kimber (1980) incorporated findings from the TRRL studies in the paper “The Capacity of Roundabouts”, where six geometric parameters were identified as having significant effect on capacity. The six key parameters were: entry width, approach half-width, effective flare length, flare sharpness, inscribed circle diameter, and entry radius. In the TRRL article, Roundabout Design For Capacity and Safety: The U.K. Empirical Methodology (U.K. Department of Transport 2007), three parameters out of the six were found to be the most relevant with regard to capacity: entry width; approach width; and flare length.

In the past decade, operational research regarding roundabouts has focused on capacity, delay, and queuing models. Capacity models used to analyze roundabouts operational performance can be categorized into gap acceptance models or linear regression models. The gap acceptance model assumes traffic entering a roundabout will do so only when an acceptable gap is found in the conflicting lane. The gap acceptance model further assumes the values for minimum acceptable gap and follow up time, the distribution of priority gaps in the flow stream, and behavior of flow on each stream.

Rodegerdts (2004) showed U.S. roundabout capacity models as a function of the circulating flow on the roundabout, follow-up headway, and critical gap in Equation 2.1.

$$Ca = \frac{v_c\, e^{-v_c t_c/3600}}{1-e^{-v_c\, t_f/3600}} \tag{2.1}$$

Where:

c_a = approach capacity vehicles per hour,

v_c = circulating flow rate vehicles per hour,

t_c = critical gap (sec), and

t_f = follow-up time (sec).

This equation estimates the capacity of a roundabout's approach (entry lanes) via input parameters such as circulating conflicting traffic volume (v_c), follow-up time (t_f), and critical gap (t_c).

Wu (2001) introduced a roundabout capacity equation as the German capacity formula in the German Highway Capacity Manual. The capacity of a roundabout is an exponential equation and it is derived from gap acceptance theory. Wu recommended that estimated capacity is a function of conflicting flow, number of lanes in roundabout entries and conflicting lanes, critical headway (4.1 sec), follow-up headway (2.9 sec), and minimum headway of circulating traffic (2.1 sec). Australia's current capacity model is based on a gap acceptance method and assumes the acceptable gap and follow up time and the conflicting flow to be constant. The assumption that the acceptable gap and follow up time are constant can lead to errors in capacity prediction under some circumstances. At low traffic flow, capacity will be overestimated and underestimated at high traffic flow.

The regression model uses different descriptive variables to predict roundabout capacity. Functional equations that relate roundabout capacity to the variables are developed using roundabout parameters. The UK model used for predicting roundabout capacity is a linear regression model based on data collected on different roundabouts in the UK over a long period of time. The formula used in this model was developed by R.M. Kimber in 1980. The model takes into account the flow characteristics and some geometric parameters of the roundabout. The UK model requires a large amount of data over a wide variety of roundabout types to accurately calculate capacity. Roundabout capacity estimation presented in the FHWA Roundabout Guide (2000) is based on the British regression model. The FHWA estimation is a simplification of the British roundabout capacity equations developed by Kimber. Kimber's equation was simplified by assuming a particular geometric design even though the equation is presented as applicable for inscribed diameters from 80 to 180 feet (ft) (24 to 55 meters (m)).

The NCHRP Report 572 uses a combined gap acceptance or linear regression model. The model is based on empirical regression from collected data on conflicting flow and follow-up headway. The equation for estimating the capacity based on the conflicting flow is shown in Equation 2.2.

$$c_{crit} = 1130e^{-0.0010v_c} \tag{2.2}$$

Where:

c_{crit} = capacity of the critical lane on the approach (vehicles per hour)

v_c = conflicting flow(vehicles per hour).

2.4 Entry Capacity

The approach width is the width of the traveled-way in advance of any entry flare. The typical approach width in the United States is 12 feet. The entry width is the width of the traveled-way at the point of entry. The FHWA (2000) identifies the entry width as the "largest determinant of a roundabout's capacity". The entry can be designed to increase capacity by either adding a full lane upstream of the roundabout or by widening the approach gradually (flaring) through the entry geometry (NCHRP 2010). The NCHRP recommends an entry width of 24 to 30 feet for two-lane entry and 36 to 45 feet for three-lane entry. It does not however, specify how far back the additional lane or flaring should begin.

In Europe, where flaring design is more common than an additional lane design, the U.K. Department of Transport Design Manual (U.K. Department of Transport 2007) recommends flare lengths of about 82 feet (25 meters) for widening to effectively increase capacity. Flare lengths greater than about 328 feet (100 meters) results in higher speed which undermines the main purpose of modern roundabout configuration.

The configuration of a modern roundabout is such that it reduces speed to improve safety and enhance traffic flow. Therefore, when increasingly long lane lengths are used, the safety benefit of roundabouts may be forfeited. The 82 foot recommendation by the U.K. Department of Transport Design Manual (U.K. Department of Transport 2007) has not been tested in the U.S., but since no data on the additional lane or flare length has been provided some state agencies follow the overseas guidelines. Interim requirements and guidance on roundabouts by the New York Department of Transportation (New York Department of Transportation 2000) suggest a flare length of 41 feet (12.5 meters) to 328 feet (100 meters) for urban areas and 66 feet (20 meters) to 325 feet (100 meters) for rural areas.

So far, there is one known model that analyzes roundabout capacity while taking into account the flare or additional lane length. Wu (1997) developed a model using probability theory to estimate the capacity of an unsignalized crossroad and T-junction intersections taking into account the length of the turn lanes. Wu determined that the flare or additional lane lengths do affect capacity of intersections and he determined the factor to account for that effect. Wu determined that for a right flared approach,

$$k_{f,right} = \frac{1}{\sqrt[n_{f,right}+1]{(x_L+x_T)^{n_{f,right}+1} + x_R^{n_{f,right}+1}}} \tag{3.3}$$

where:

$k_{f,right}$ = factor for estimating the capacity of a shared lane

$n_{f,right}$ = length of queue space in number of vehicles

x_L= degree of saturation, left-turning traffic stream

x_T = degree of saturation, through traffic stream

x_R= degree of saturation, right-turning traffic stream

The findings from Wu's model were used in the FHWA 2000 to estimate the capacity of roundabouts with flared or additional lane. By dropping some subscripts and assuming that the capacities and flows in each lane are the same (that is, the entries are constantly fed with vehicles), the factor for estimating the capacity of a shared lane was estimated as:

$$k = \frac{1}{x*^{n+1}\sqrt{2}} \tag{2.4}$$

with $x_{LT} = x_R$.

By assuming flow in each lane equal to q_i and $q = q_1 = q_2$, capacity q_{max} was then estimated as :

$$q_{max} = k\sum q_i = \frac{2q}{x*^{n+1}\sqrt{2}} \tag{2.5}$$

Where q_{max} is the capacity of an entry at a double-lane roundabout, the capacity of each entry lane is then $q_{max2}/2$ which is equal to the flow, q, divided by the degree of saturation, x.

$$q_{max} = \frac{q_{max2}}{{}^{n+1}\sqrt{2}} \tag{2.6}$$

Wu (2006) points out that the exit cannot be less than the entry capacity if the full potential of the entry is to be utilized. Wu (2006) was able to identify the effect of entry length but the effect of the additional lane length at the exit was not mentioned.

Wu also assumes that the capacities of both lanes are identical and the traffic flows in both lanes at the entry are equally distributed. However, studies conducted on some double lane roundabouts in the U.S. by the NCHRP 572 shows that the right lane is utilized more frequently than the left lane and the right lane is usually considered to be the critical lane. For instance, data obtained from Kittelson & Associates on one of the double lane roundabouts in Brattleboro, Vermont showed that the right lanes had about 70% of the entry total flow, so capacity in the Wu model could be overestimated. This research tries to examine the effect of the flare/additional lane length on roundabout operation using typical U.S. driving behavior where the right lane is considered the critical lane and is utilized more frequently than the left lane.

2.5 VISSIM

In order to mimic typical U.S. driving behavior, the VISSIM microsimulation modeling software is used for analysis purposes. VISSIM is a model developed in Germany, where vehicles are modeled using parameters such as driver behavior, vehicle speeds, and vehicle type (PTV 2010). The basic traffic model ruling the movement of vehicles was developed by Rainer Wiedemann in 1974 at Karlsruhe University. It is a car-following model that considers physical and psychological aspects of the drivers. VISSIM has the ability to control gaps and headways on a lane-by-lane basis to more accurately replicate these types of operations present at roundabouts. Numerous studies have used VISSIM to examine roundabout performance due to its unique ability to mimic real world traffic operations. Trueblood et al. (2003) considered VISSIM to be a very effective micro simulation software package for roundabout performance analysis. Because of this, Trueblood and Dale used VISSIM to model existing roundabouts in the state of Missouri, and this micro-simulation software package was found to provide accurate results in roundabout performance analysis. Bared et al. (2009) used VISSIM to model roundabouts for various ranges of circulating and entry traffic volumes. They found that simulation results from VISSIM were significantly lower than from the SIDRA analytical and RODEL empirical models and were similar to field measured data used in NCHRP 572.

2.5.1 Car Following Behavior

The car following model in VISSIM is based on the continued research of Wiedemann. Details on the model are presented in research by Wiedemann et al (1991) and Fellendorf et al (2001). The basic premise of the Wiedemann model states that a vehicle is in one of four states of car following; free, approaching, following, or braking. The first state of the car following model identifies a vehicle in a free driving arrangement that does not need to respond to the performance of other vehicles; it responds only to regulatory measures such as traffic signs. At a point while driving, the distance is reduced so that the rear vehicle acknowledges the existence of the leading vehicle that it is approaching. Once the trailing vehicle has caught up to the leading vehicle, the trailing vehicle drives in a responsive manner to the performance of the vehicle in front. A distance from the leading vehicle is maintained but does continue to oscillate due to subtle changes in speed, acceleration and deceleration. A desired safe distance is preserved between the leading and the trailing vehicle. However, if the trailing vehicle moves too close to the leading vehicle, it enters the

“Braking” stage. It is at this stage where accidents are more likely to occur. Figure 2:2 shows a graphical description of the Wiedemann car following model.

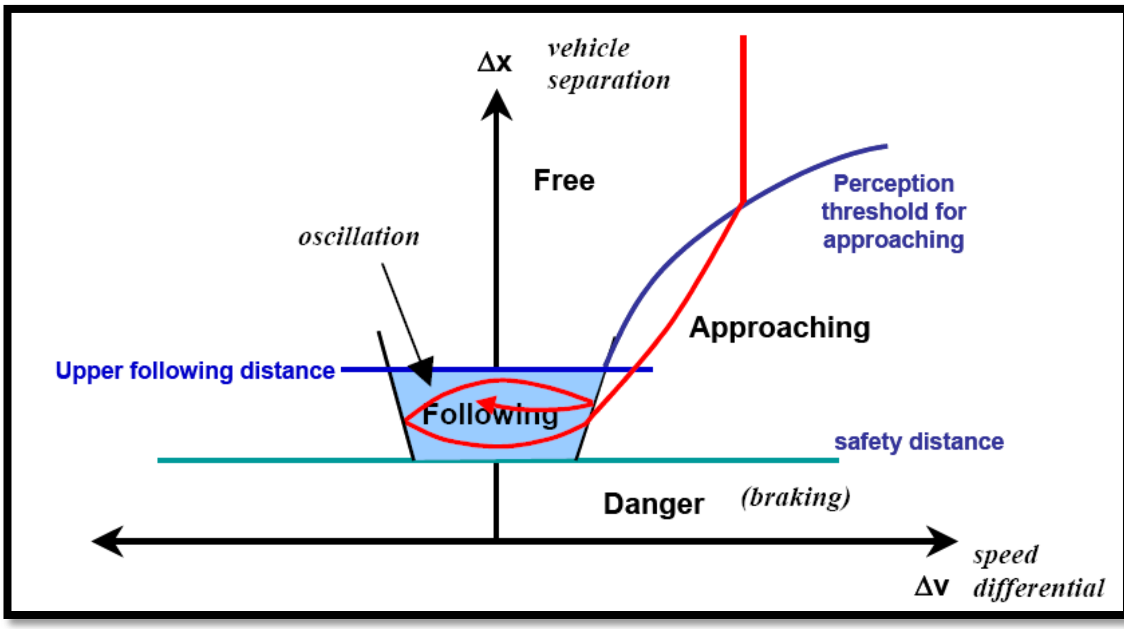

Figure 2:2 Wiedemann Car Following Logic

The Wiedemann 99 car following model was developed in 1999 to provide greater control of the car following characteristics for freeway modeling in VISSIM. The Wiedemann 99 model consists of ten calibration parameters, all labeled with a ‘CC” prefix. Each of the parameters controls a unique aspect of the car following model. The ‘CC’ parameters are categorized by how they affect the car following thresholds for Dx, car following thresholds for Dv, and acceleration parameters. Table 2:2 provides a description and the default values for each of the ‘CC’ parameters associated with the Wiedemann 99 model.

Category	VISSIM Code	Description	Default Value
Thresholds for Dx	CC0	**Standstill distance:** Desired distance between lead and following vehicle at v = 0 mph	4.92 ft
	CC1	**Headway Time:** Desired time in seconds between lead and following vehicle	0.90 sec
	CC2	**Following Variation:** Additional distance over safety distance that a vehicle requires	13.12 ft
	CC3	**Threshold for Entering 'Following' State:** Time in seconds before a vehicle starts to decelerate to reach safety distance (negative)	-8.00 sec
Thresholds for Dv	CC4	**Negative 'Following' Threshold:** Specifies variation in speed between lead and following vehicle	0.35 ft/s
	CC5	**Positive 'Following Threshold':** Specifies variation in speed between lead and following vehicle	0.35 ft/s
	CC6	**Speed Dependency of Oscillation:** Influence of distance on speed oscillation	11.44
Acceleration Rates	CC7	**Oscillation Acceleration:** Acceleration during the oscillation process	0.82 ft/s^2
	CC8	**Standstill Acceleration:** Desired acceleration starting from standstill	11.48 ft/s^2
	CC9	**Acceleration at 50 mph:** Desired acceleration at 50 mph	4.92 ft/s^2

Table 2:2 Wiedemann 99 Parameters

(Source: VISSIM 5.30 Manual, PTV AG, Karlsruhe, Germany)

Another important parameter related to the car following behavior in VISSIM is the number of time steps per second. VISSIM allows for the user to choose from one to ten time steps per second while running the simulation. Increased time steps per second provide more accurate results of the simulation. Utilizing a lower time step per second introduces the potential for overcompensation by vehicles.

2.5.2 Necessary Lane Changing Behavior

A necessary lane change is defined in VISSIM as lane change that is necessary for a vehicle to reach its final destination in the network. VISSIM lane changing behavior is characterized by maximum and accepted deceleration rates for the merging (own) and trailing vehicle. Driver aggressiveness can be controlled by modifying the maximum and accepted deceleration rates as well as the reduction rate of the deceleration value as the vehicle approaches its merge point (PTV 2010).

VISSIM also allows the modeler to specify the general lane driving behavior of the model. VISSIM has two options for the lane driving behavior, right-side rule or free lane selection. The right-side rule allows overtaking of other vehicles in the left lane with restrictions, and free lane selection allows overtaking of other vehicles in any lane (PTV 2010).

Other parameters related to the necessary lane changing behavior include the emergency stop distance and the waiting time before diffusion. The emergency stop distance is the distance before a destination connector that a vehicle will stop and wait for a gap to merge. The waiting time before diffusion defines the maximum time that a vehicle will wait at its emergency stop distance before it will be removed from the network (PTV 2010).

2.5.3 Lane Changing Distance

The lane change distance in VISSIM is a connector and routing decision based parameter. It defines the distance behind a destination connector that a vehicle will start to search for a lane change to reach that connector. In order for the lane change distance to utilize its full value, a vehicle must pass the start of the destination routing decision at a point that is equal to or greater than the lane change distance. Otherwise, the vehicle will only start searching for a lane change at the point that it passes the start of the destination routing decision.

CHAPTER 3 : METHODOLOGY

This section details how the research effort was conducted with respect to modeling the impact of additional lane length on roundabout operation. To model the full additional lane design as shown in Figure 1:3, a full lane is added on the right side at the entry with a taper of sufficient length to enable vehicles to diverge into the additional lane. In flare design, a single lane is gradually widened into two lanes at the entry. Both design cases result in the widening of the entry to increase the rate at which vehicles can potentially enter the roundabout at a given time. This means that in terms of operation, a single traffic stream separates for both the additional and flare design into two streams. The additional lane design was used in this research to examine the effect on roundabout performance.

Depending on the design requirement, flaring can also allow for more than one vehicle stream at the yield point. Both design cases result in the widening of the entry to increase the rate at which vehicles enter the roundabout at a given time. This means that in terms of operation they are similar, if not the same. The main principle concerning the flaring design relies on widening the approach gradually through the entry geometry as shown in Figure 1:4. Both design cases result in the widening of the approach to increase the rate at which vehicles enter the roundabout. In both additional lane and flaring, a single traffic stream separates into two streams. This justified the use of the additional lane design in this research to examine the effect of the additional lane length on a roundabout with findings applied to flared entry as well.

A literature review was carried out to identify elements and factors that influence the operation of roundabouts, flaring and adding full lane. The pertinent literature is reviewed in six sections. The first section presents a general knowledge on roundabout and its features; the second section examines issues pertaining to roundabout operational performance; the third section takes a closer look at the entry capacity of roundabout; the fourth section presents a description of traffic simulation car-following model; and the fifth section examines the widely-used microscopic traffic simulation model VISSIM, while the last section summarizes the findings from the literature review and how they assure the necessary competency of the methodology and findings of this study.

A hypothetical double lane roundabout with four legs was first examined in VISSIM under varying additional lane lengths at the entry and exit. Zero feet (single lane entry and exit) length variations were included even though such scenario is not practical; it was included to illustrate the relationship between delay and the length up to zero. For comparison purposes, similar variations were then tested on an existing double lane roundabout with data from NCHRP 572. Before testing the variations on an existing roundabout, the model was calibrated. Calibration was performed to ensure that the model correctly predicted traffic performance to help in accepting or rejecting the hypotheses stated earlier. Calibration effort requires Field data or other validated analytical models are required for calibration. Due to the lack of validated analytical models, the hypothetical model was analyzed using VISSIM default values with average of the measure of effectiveness calculated within an acceptable level of confidence. The existing model was calibrated against field data obtained from the NCHRP 572 report. To ensure accurate comparison, an existing roundabout with geometric features and operational performances similar to the hypothetical model was chosen.

In order to layout the roundabout correctly in VISSIM, guidelines by (Trueblood et al.2003), and (Li et al.2013) were used. From both studies, the techniques of placing the reduced areas at the conflicting sections were adapted. The reduced speed areas were kept at a length of 17 feet and placed at 8 feet from the yield line on each lane of the approach. Reduced speed areas were also placed in the circulatory roadway at a length of 17 feet right before the entry areas. Travel speeds of 20 miles per hour were used in the reduced speed zones as recommended by Trueblood and Dale (2003). Since VISSIM is a stochastic model whose results vary depending on the random seed number used, the model was run multiple times and the average results were used. For this study, multiple simulations were made for each scenario with a running time of one hour.

Both models were analyzed in VISSIM using a calculated number of simulation runs under different scenarios. Table 3:1 shows the different scenarios used for model analysis. The length variations carried out in this research were grouped into scenarios:

Scenario 1: Only the entry additional lane length was varied while the exit additional lane length was kept at zero (single exit).

Scenario 2: Both entry and exit additional lane length were varied.

Under Scenario 1, three variations were considered:

1. Additional lane lengths at the entry at all four legs are varied.
2. An additional lane at the entry with the maximum volume is varied.
3. An additional lane at the entry with the least volume is varied.

Under Scenario 2, three variations were considered:

1. Additional lane lengths at the entry and exit at all four legs are varied at the same time.
2. An additional lane at the entry and exit with the maximum volume is varied at the same time.
3. Only one additional lane at the entry and exit with the least volume is varied at the same time.

	Hypothetical Model Scenario 1								**Hypothetical Model Scenario 2**							
Additional Lane	East		West		South		North		East		West		South		North	
Location	Entry	Exit	Entry	Exit	Entry	Exit	Entry	Exit	Entry	Exit	Entry	Exit	Entry	Exit	Entry	Exit
Variation 1	X		X		X		X		X	X	X	X	X	X	X	X
Variation 2					X								X	X		
Variation 3			X								X	X				
	Existing Model Scenario 1								**Existing Model Scenario 2**							
Additional Lane	East		West		South		North		East		West		South		North	
Location	Entry	Exit	Entry	Exit	Entry	Exit	Entry	Exit	Entry	Exit	Entry	Exit	Entry	Exit	Entry	Exit
Variation 1	X		X		X		X		X	X	X	X	X	X	X	X
Variation 2					X								X	X		
Variation 3							X								X	X

Table 3:1 Model Scenarios

For the hypothetical model, VISSIM default values for headway were used. Data collection points used for capturing delay data in VISSIM for the hypothetical model were placed at similar locations specified in the NCHRP 572 so as to be able to compare results. In the NCHRP Report 572, speed, flow, service time, travel time and delay data were collected at the following locations shown in Figure 3:1:

- upstream of the roundabout about 250 feet from the yield line (u)
- entry yield line (y)
- midpoint of the splitter island (s)
- exit from the circulatory roadway (e)

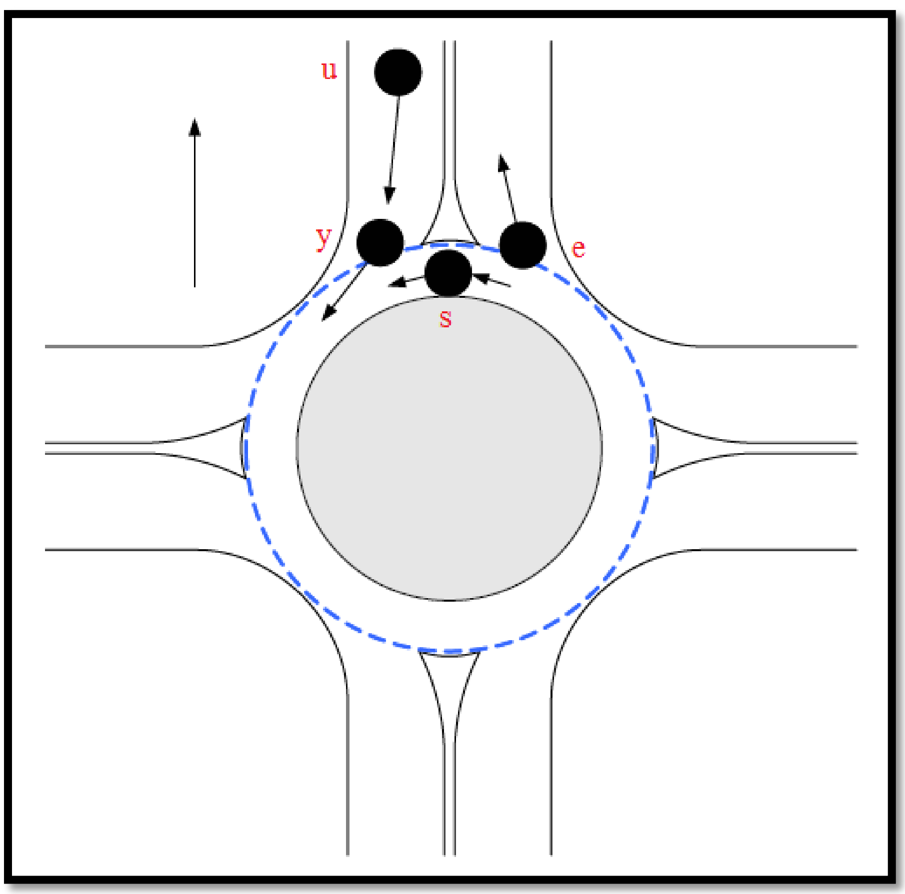

Figure 3:1 Data Collection Locations Used in NCHRP Report 572

In this research, the travel time sections in VISSIM were placed at 250 feet from the yield line on the approach and the exit where the vehicles exit the circulatory roadway. This allowed the software to compute the delay in travel 250 feet from the yield line on the approach to the point where a vehicle exits the circulatory roadway. The existing roundabout used for comparison was set up in VISSIM with data collection points placed at similar locations as those used in the NCHRP 572. The model was then calibrated using field data from NCHRP 572. The calibration effort begun with the VISSIM default values and gradually adjusting the reduced speed, driving behavior, yield bar placement, headway, and minimum gaps until the measured field travel time data closely matched the VISSIM data. The field travel time data was the same data used in the NCHRP 572 that was obtained from Kittelson Associates.

3.1 Research Hypotheses

Three hypotheses for this research are evaluated:

Hypothesis H1: **Shorter additional lane lengths are more effective in reducing delay than longer lengths**. The national data on roundabout points out that roundabout delay can be decreased by either adding a full lane upstream of the roundabout or by widening the approach gradually (flaring) through the entry geometry (NCHRP 2010), but it does not give any guidelines on the length of the additional lane. This hypothesis aims to address the question of whether shorter additional lane lengths are more effective in reducing delay than longer lengths.

Hypothesis H2: **Adjusting the additional entry lane length should be done concurrently with the exit lane length in order to reduce delay.** Earlier roundabout researchers have suggested having a balanced entry and exit capacity in order to avoid bottleneck effect. But they focused on the number of entry lanes and not the length of the additional lane. It is not clear if balancing the entry and entry and exit capacities involves balancing the entry and exit lane lengths as well. This hypothesis aims to address the question of whether increasing the additional lane length has to be done with increasing exit lane length in order to reduce delay.

Hypothesis H3: **Adjusting the additional lane length on all legs is more effective in reducing delay than adjusting just one leg.** This hypothesis aims to address the question of whether increasing the additional lane length on all legs of the roundabout is more effective in reducing delay than increasing one leg.

CHAPTER 4 : MODEL DEVELOPMENT, ANALYSIS, AND EVALUATION

A hypothetical model was first developed in VISSIM to study the general operational effect of the additional lane. Findings from the hypothetical model are then compared with that of an existing roundabout. The hypothetical double lane roundabout with four legs was first examined in VISSIM under varying additional lane lengths at the entry and exit. For comparison purposes, similar variations were then tested on an existing double lane roundabout with data from NCHRP 572.

For each model, the five simulation runs were initially executed using different random number seeds. The actual seed values for each run were documented so that the results could be replicated later. Reporting the average results of multiple runs was necessary due to the stochastic nature of the model, but in order to ensure that the value reported was a true statistical representation of the average, the following formula for a 95 percent confidence interval was applied:

$$N = \left(2 * t_{0.025,N-1} \frac{s}{R}\right)^2 \tag{4.1}$$

where:

R = 95-percent confidence interval for the true mean

$t_{0.025,N-1}$ = Student's t-statistic for two-sided error of 2.5 percent (totals 5%) with N-1 degrees of freedom

s = standard deviation of about the mean for selected MOE

N = number of required simulation runs

This formula was used to determine the minimum number of runs needed to achieve a 95% confidence interval after the initial data set was generated using the 5 multiple runs. Several factors were considered for selecting the representative case study. The most important factors were:

1. Four leg
2. Two lane
3. Flare or additional lane entry
4. MOE data availability
5. Closeness for field visit

One such roundabout was identified: the Brattleboro Roundabout at the intersection of Route 9 and Route 5 in Brattleboro, Vermont. The Brattleboro Roundabout is a four leg roundabout with the legs aligned at ninety degrees. It is a two lane roundabout with different additional lane lengths.

Figure 4:1 Brattleboro Double Lane Roundabout

4.1 Hypothetical Double Lane Roundabout Model Development and Analysis

The roundabout (Figure 4:2) used in this study was designed in AutoCAD with a focus on the six important parameters given by TRL (U.K. Department of Transport 2007). The design was based on the guidelines in the NCHRP Report 672. The roundabout had two circulatory lanes and four legs with single lanes that diverged into two lanes at the entry and merged into one at the exit. An inscribed circle of 180 feet was used for this study. The model had the four approaches aligned at 90 degrees. The AutoCAD layout was subsequently uploaded into VISSIM.

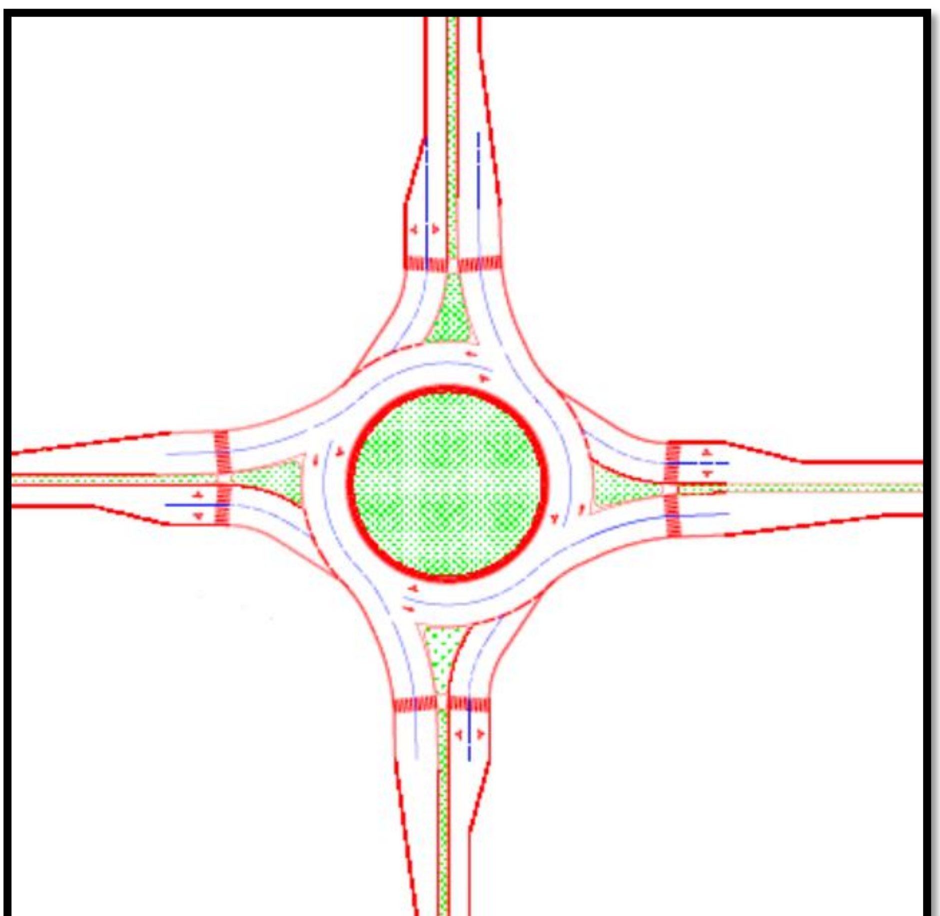

Figure 4:2 Hypothetical Roundabout Design

For the purpose of this analysis, no specific volume was assigned on lane basis. Vehicles were allowed to freely choose lanes but the links and diving behavior were configured such that the right lanes would be used more frequently (about seventy percent usage was observed from simulation). This allowed the roundabout model to operate with driving behavior similar to real life driving behavior were vehicles are free to change lanes when prevailing conditions are not favorable. A quarter of the traffic made right and left turns and one half proceeded straight through past the roundabout. These turns were made by freely choosing either the left or right lanes depending on downstream conditions but the right lane was used most of time during less delays and short queues. A degree of saturation less than 0.80 was targeted based on the following assumptions:

- The major road traffic is associated with North and South - movements with a volume of 800 vehicles per hour in each direction, - East-west movements were on the minor road with the same volume of 350 vehicles per hour in each direction. Tables 4:1 and 4:2 indicate the total vehicle volumes and the associated turning movements for each entry flow.

- The right lane was assumed to be the critical lane in both movements

- Fifteen percent of all demand volumes consisted of heavy vehicles

DIRECTION	ENTRY FLOW (veh/hr)
Eastbound (EB)	350
Westbound (WB)	350
Northbound (NB)	800
Southbound (SB)	800

Table 4:1 Hypothetical Model Entry Flow

MOVEMENT	EB	WB	NB	SB
THROUGH (veh/hr)	150	150	400	400
LEFT TURN (veh/hr)	100	100	200	200
RIGHT TURN (veh/hr)	100	100	200	200
U-TURN (veh/hr)	0	0	0	0

Table 4:2 Hypothetical Model Turning Movements

	EB LL	EB RT	WB LL	WB RT	NB LL	NB RT	SB LL	SB RT
ENTRY VOLUME , v_i (veh/hr)	**105**	**245**	**105**	**245**	**240**	**560**	**240**	**560**

Table 4:3 Hypothetical Model Entry Volume Lane Distributions

Using the 2010 HCM roundabout analysis, the entry volumes were first adjusted for heavy vehicles assuming 15 percent of the traffic was heavy vehicle. The Heavy adjustment factor, f_{HV} was computed using Equation 4.2.

$$f_{HV} = \frac{1}{1+P_T(E_T-1)} \quad (4.2)$$

where:

P_T = proportion demand volume that consist of heavy vehicle

E_T = passenger car equivalent for heavy vehicles

In this study, P_T was assumed to be 0.15 (assuming 15 percent of entry volume consists of heavy vehicle). E_T was assumed to be 2.0 (from 2010 HCM manual, see

Appendix D). The demand flow rate in passenger car equivalent was calculated using Equation 4.3.

$$v_{i,pce} = \frac{v_i}{f_{HV}} \tag{4.3}$$

where :

$v_{i,pce}$ = demand flow rate in passenger car equivalent (pc/h)

v_i = demand volume (veh/h)

f_{HV} = heavy vehicle adjustment factor

Table 4.4 shows the entry volume in passenger car equivalent after being adjusted for heavy vehicles.

	EB LL	**EB RL**	**WB LL**	**WB RL**	**NB LL**	**NB RL**	**SB LL**	**SB RL**
ENTRY VOLUME, $v_{i,pce}$ (pc/h)	121	282	121	282	276	644	276	644

Table 4:4 Entry Volume Lane Distribution Adjusted for Heavy Vehicles

Using Equation 4.4, the capacity of the right lane which was considered the critical lane was computed.

$$c_{e,R,pce} = 1{,}130e(-0.7 \times 10^{-3})v_{pce} \tag{4.4}$$

where :

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

v_{pce} = conflicting flow, pc/h

Table 4.5 shows the capacity of the critical lane (right lane) adjusted for heavy vehicles.

	EB LL	EB RL	WB LL	WB RL	NB LL	NB RL	SB LL	SB RL
CRITICAL LANE		X		X		X		X
CRITICAL LANE CAPACITY, $c_{e,R,pce}$ (pc/h)		692		692		825		825

Table 4:5 Hypothetical Model Critical Lane Capacity

Using Equation 4.5, the v/c ratios were determined for each critical lane (Table 4.6)

$$\frac{v}{c} = \frac{v_{i,pce}}{c_{e,R,pce}} \tag{4.5}$$

	EB LL	EB RL	WB LL	WB RL	NB LL	NB RL	SB LL	SB RL
v/c RATIO		0.41		0.41		0.78		0.78

Table 4:6 Hypothetical Model v/c Ratio

The v/c ratio for the north and southbound leg critical lane was 0.78 and 0.41 for the east and westbound leg critical lane was 0.41 using the analytical method presented in the HCM.

Starting with additional lane length of zero (single lane entry and exit), the roundabout operational performance was analyzed in VISSIM for the five initial simulation runs. Then using Equation 4.6 the minimum number of runs needed to achieve a 95% confidence interval was computed. For each model, the 5 simulation runs were initially executed using different random number seeds with different random number seeds. The actual seed values for each run were documented so that the results could be replicated later. Reporting the average results of multiple runs was necessary due to the stochastic nature of the simulation model, but in order to ensure that the value reported was a true statistical representation of the average, Equation 4.1 was applied.

This formula was used to determine the minimum number of runs needed to achieve a 95% confidence interval after the initial data set was generated using the 5 multiple runs. The additional lane lengths were analyzed in VISSIM for the two scenarios. The zero foot additional lane model, (single lane) was first analyzed VISSIM to generate the initial data using the 5 multiple runs. The average delay for the initial five simulation runs are shown in Table 4.7. The descriptive statistics for five runs are shown in Table 4.8.

Runs	**Intersection Delay (s)**
1	9.1
2	8
3	10
4	9.1
5	7.7

Table 4:7 Hypothetical Model Initial Simulation Delay Data

Intersection Delay (s)	
Mean	8.78
Standard Error	0.42
Median	9.1
Mode	9.1
Standard Deviation	0.93
Sample Variance	0.87
Kurtosis	-1.40
Skewness	0.08
Range	2.3
Minimum	7.7
Maximum	10
Sum	43.9
Count	5

Table 4:8 Hypothetical Model Descriptive Statistics

From the t Distribution table (Appendix D), the student's t-statistic for two-sided error of 2.5 percent (totals 5 percent) with N-1 degrees of freedom was found to be 2.571. Using Equation 4.6, the student's t-statistic of 2.751 (for two-sided error of 2.5 percent, with N-1 degrees of freedom) from the t-distribution table and parameters from the Table 4.8, the minimum required number of runs was computed as:

$$N = \left[2 * (2.751) * \left({}^{0.931128348}/_{1.156148547}\right)\right]^2$$

$$N = 17.15 \approx 17\ RUNS$$

After running a few scenarios with additional lane lengths at 0 feet, 150 feet, 250 feet, 350 feet, 450 feet and 550 feet.with 17 simulation runs, t-tests were conducted that indicated that indicated no significant statistical difference in the means of the data, so the remaining work was continued with five simulation runs, and the reporting is based on that. This corresponds with work of many researchers who use at least five simulation runs per scenario in their simulation modeling efforts. The VISSIM lane closure feature was utilized to make the zero foot length possible. Reducing the exit and entry lanes on a double lane roundabout to single lanes is not practical; it was done in this study only to illustrate the extent of the delay effect of no change.

The length variation consisted of two scenarios (see Table 2.1):

Scenario 1: Only the entry additional lane length was varied while the exit additional lane length was kept at zero (single exit).

Scenario 2: Both entry and exit additional lane length were varied.

Under Scenario 1, three variations were considered:

1 Additional lane lengths at the entry at all four legs are varied. This scenario was represented by HA in this study, where H represents the hypothetical model and A represents all legs.

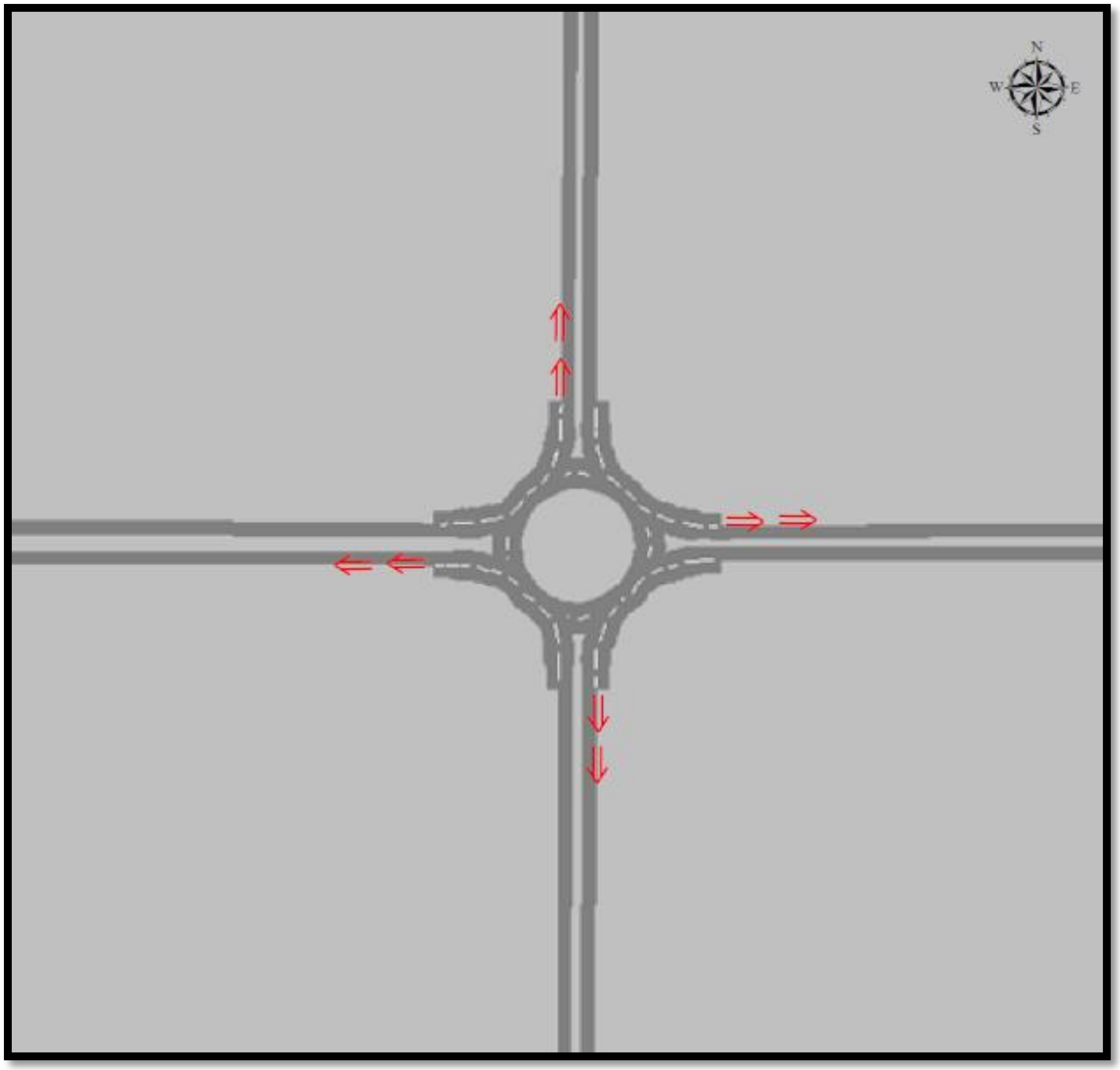

Figure 4:3 HA Scenario Variations

2 An additional lane at the entry with the maximum volume (south leg) is varied. This scenario was represented by HS, where H represents the hypothetical model and S represents south leg.

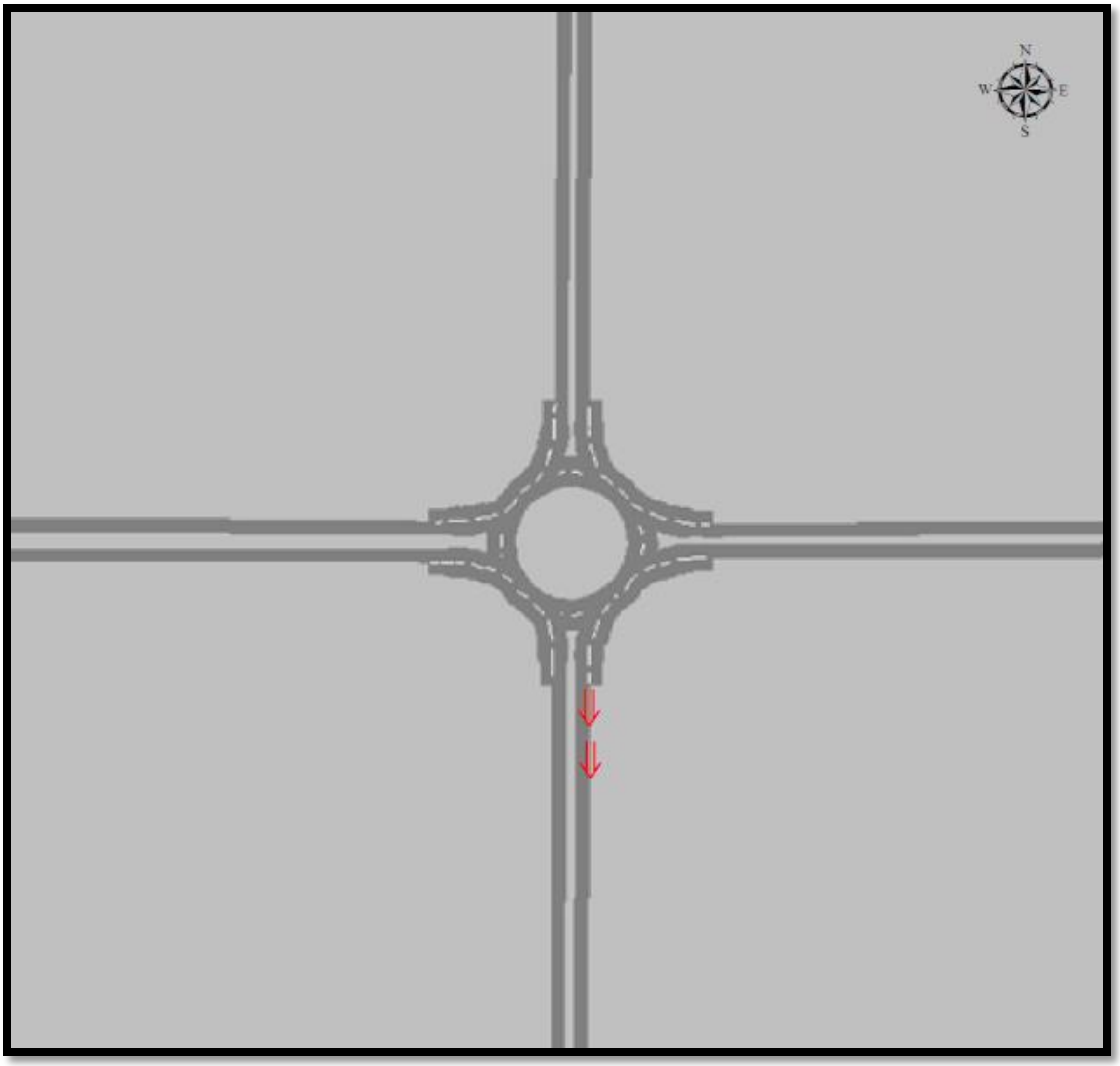

Figure 4:4 HS Scenario Variations

3 An additional lane at the entry with the least volume (west leg) is varied. This scenario was represented by HW, where H represents the hypothetical model and W represents leg.

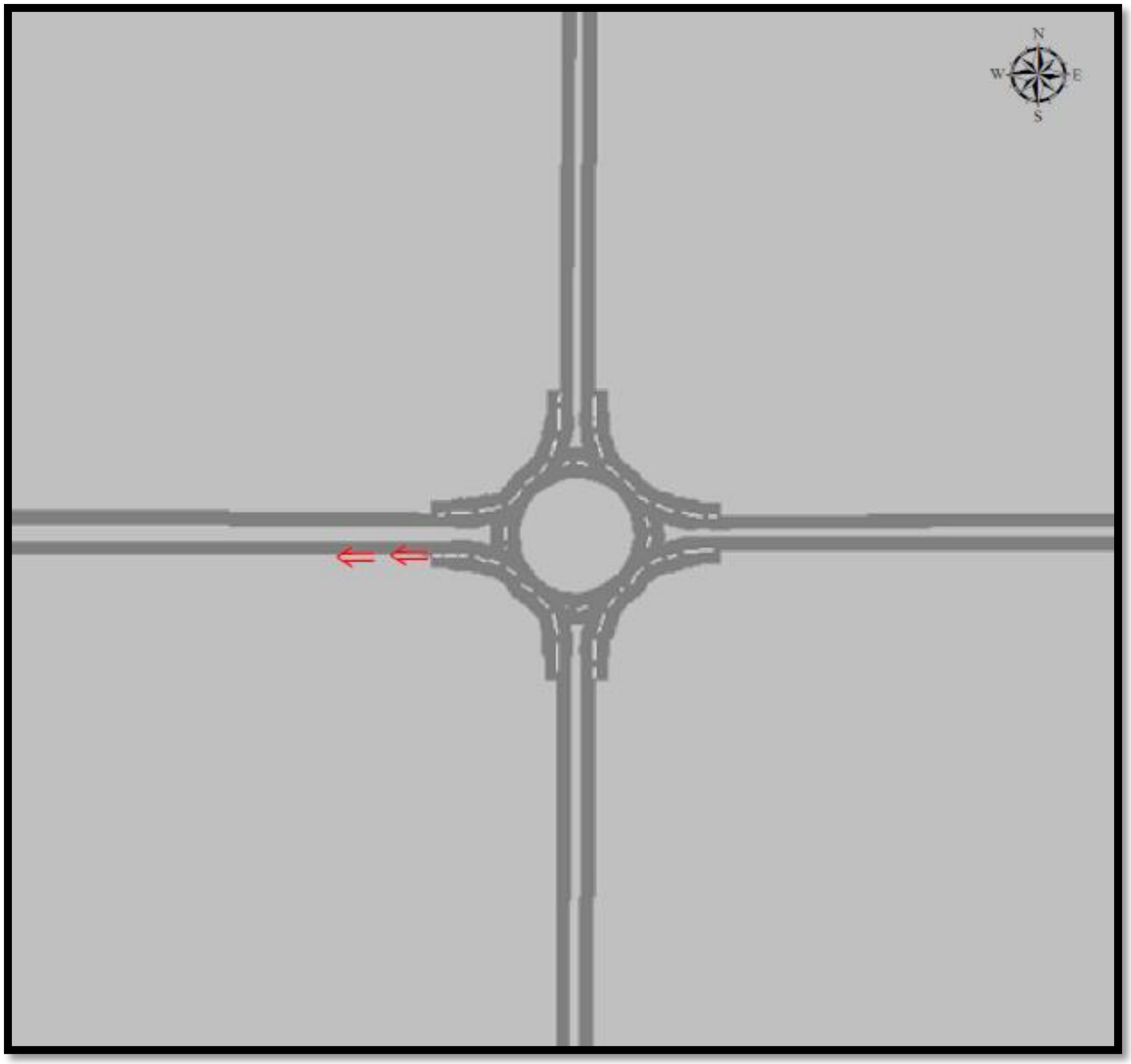

Figure 4:5 HW Scenario Variations

Under Scenario 2, three variations were considered:

1. Additional lane lengths at the entry and exit at all four legs are varied at the same time. This scenario was represented by HAX in this study, where H represents the hypothetical model, A represents all legs and X represents exit.

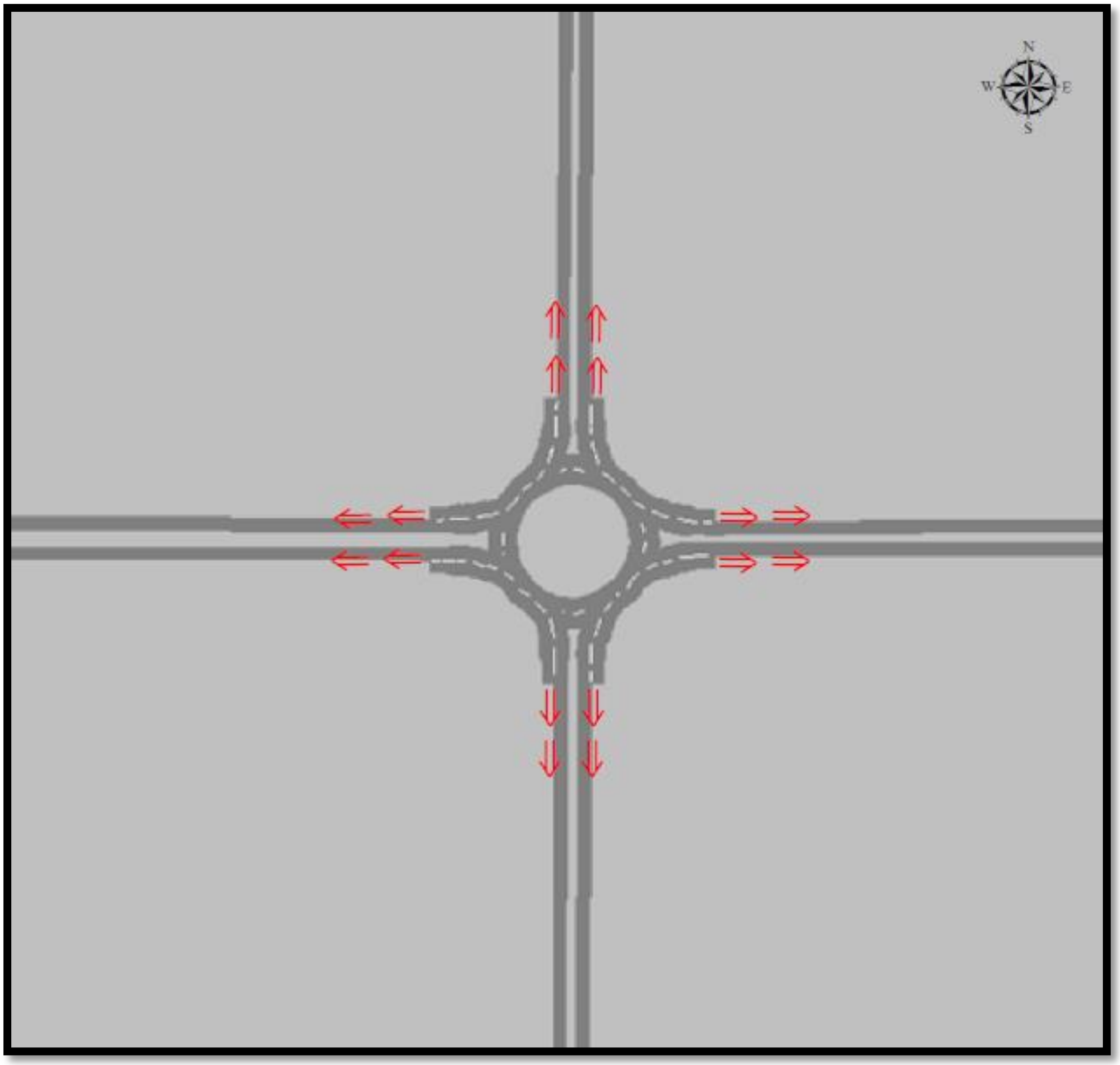

Figure 4:6 HAX Scenario Variations

2. An additional lane at the entry and exit with the maximum volume (south leg) is varied at the same time. This scenario was represented by HSX, where H represents the hypothetical model, S represents south leg and X represents exit.

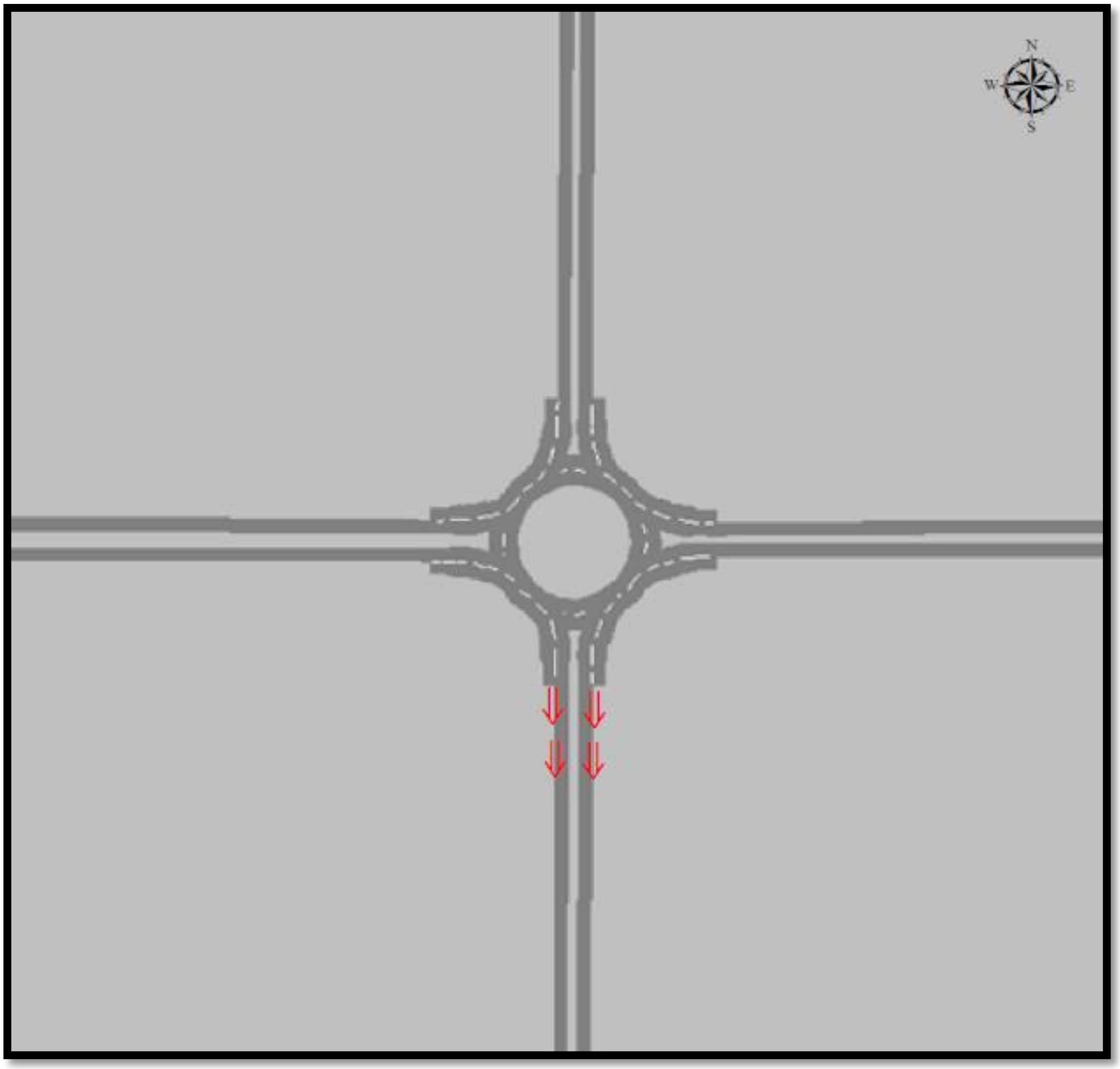

Figure 4:7 HSX Scenario Variations

3. Only one additional lane at the entry and exit with the least volume (west leg) is varied at the same time. This scenario was represented by HWX, where H represents the hypothetical model, W represents west leg and X represents exit.

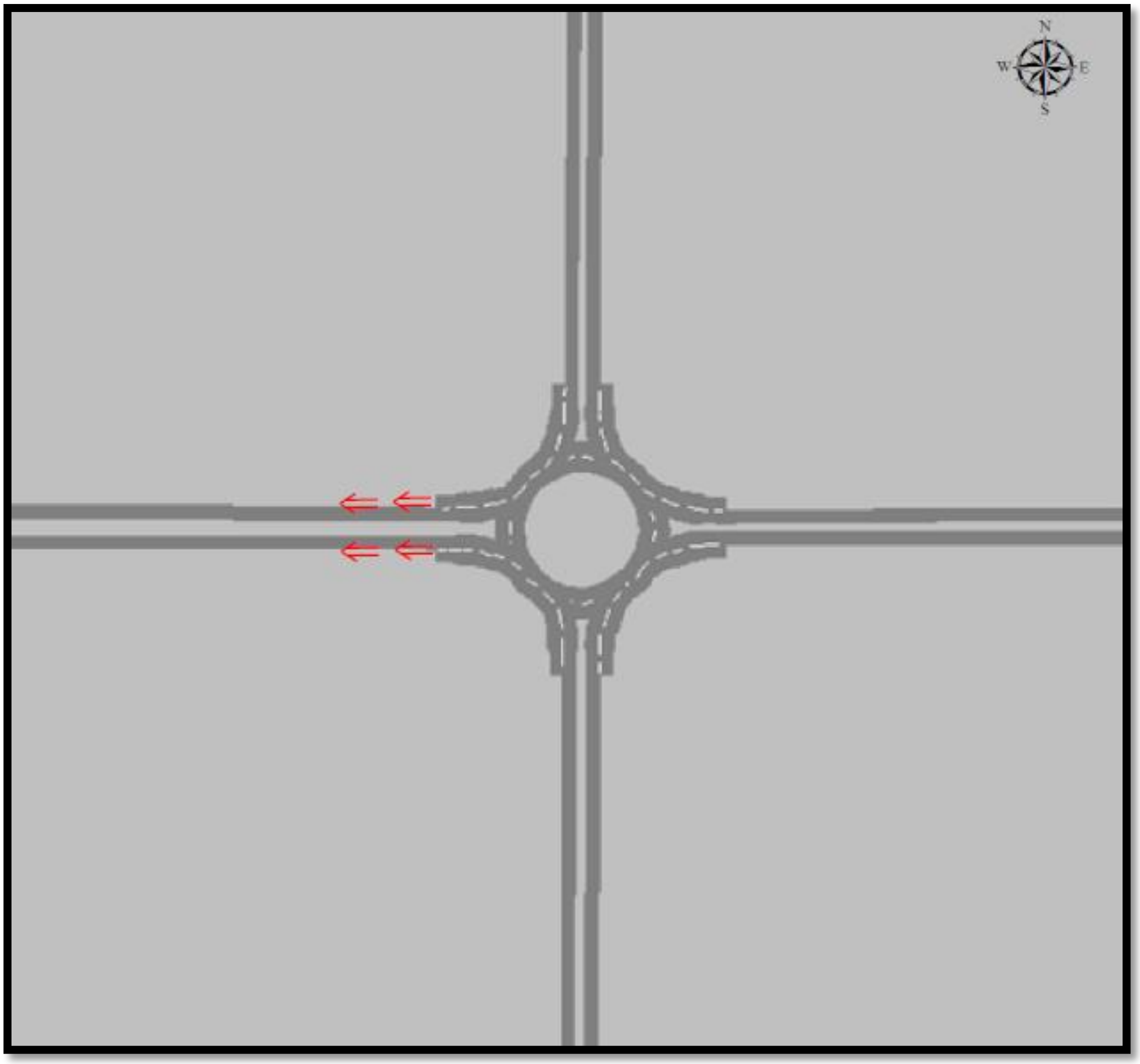

Figure 4:8 HWX Scenario Variations

4.2 Existing Double Lane Roundabout Model Development

The roundabout chosen for this analysis was the Brattleboro roundabout in Vermont. This is one of the roundabouts that the NCHRP 572 collected data on to study the roundabout operations in the United States. The data from the NCHRP 572 study was used to calibrate and validate the model in VISSIM. The Brattleboro roundabout has similar configuration to the hypothetical model used in this study. It is a double lane roundabout with four legs aligned at 90 degrees. Its inscribed circle diameter is 176 feet and all legs have additional lane lengths greater than 100 feet. Figure 4:9 which is the latest drawing of the roundabout obtained from Vermont Transportation Agency shows the different additional lane lengths on the approach. The south, east and north legs have exceptionally long taper lengths and these lengths were included in the model set up. The figure also shows new pavement markings where three lanes have been proposed for the northbound entry. This study used the exiting configuration (two lane entries) at the time the field data was collected for the NCHRP 572 (NCHRP Report 572 2007).

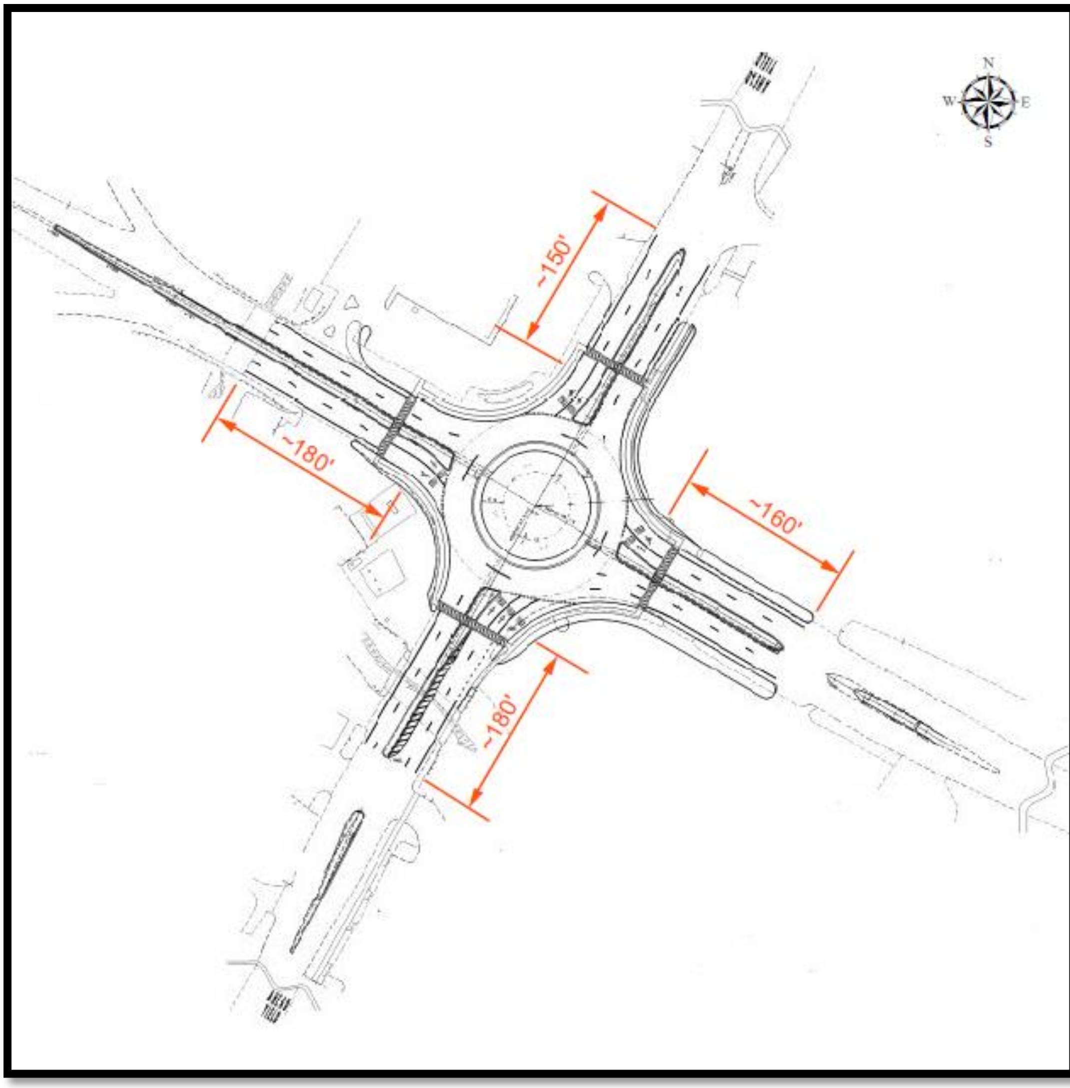

Figure 4:9 Brattleboro Roundabout Design

(Source: Vermont Transportation Agency)

The field data collection determined that the volumes for east, west, south and northbound legs were 832 veh/hr, 441 veh/hr, 515 veh/hr and 1051 veh/hr, respectively (Table 4:9). Table 4:10 shows the turning movement for the entry flows.

DIRECTION	ENTRY FLOW (veh/hr)
Eastbound (EB)	832
Westbound (WB)	441
Northbound (NB)	1051
Southbound (SB)	515

Table 4:9 Existing Model Entry Flow

MOVEMENT	EB	WB	NB	SB
THROUGH (T)	254	192	397	320
LEFT TURN (L)	204	174	330	67
RIGHT TURN (R)	343	75	299	117
U-TURN (U)	31	0	25	11

Table 4:10 Existing Model Turning Movement

	EB LL	EB RL	WB LL	WB RL	NB LL	NB RL	SB LL	SB RL
ENTRY VOLUME , v_i (veh/hr)	212	620	156	285	232	818	106	409

Table 4:11 Existing Model Entry Volume Lane Distributions

Using the 2010 HCM roundabout analysis, the entry volumes were first adjusted for heavy vehicles assuming 15 percent of the traffic comprised heavy vehicle. The Heavy vehicle adjustment factor, f_{HV} was computed using Equation 4.2. In this study, P_T was assumed to be 0.15 for the existing model as well. E_T was assumed to be 2.0 (from 2010 HCM manual, see Appendix D). The demand flow rate in passenger car equivalent was calculated using Equation 4.3. Table 4.12 shows the entry volume in passenger car equivalent after being adjusted for heavy vehicles.

	EB LL	EB RL	WB LL	WB RL	NB LL	NB RL	SB LL	SB RL
ENTRY VOLUME, $v_{i,pce}$ (pc/h)	244	713	180	328	267	941	122	470

Table 4:12 Entry Volume Lane Distribution Adjusted for Heavy Vehicles

Using Equation 4.4, the capacity of the right lane which was considered the critical lane was computed. Table 4.13 shows the capacity of the critical lane (right lane) adjusted for heavy vehicles. Table 4.14 shows the v/c ratios using Equation 4.5.

	EB LL	EB RL	WB LL	WB RL	NB LL	NB RL	SB LL	SB RL
CRITICAL LANE		X		X		X		X
CRITICAL LANE CAPACITY, $c_{e,R,pce}$ (pc/h)		744		562		668		760

Table 4:13 Existing Model Critical Lane Capacity

	EB LL	EB RL	WB LL	WB RL	NB LL	NB RL	SB LL	SB RL
v/c RATIO		0.96		0.58		1.41		0.62

Table 4:14 Existing Model v/c Ratio

Using the HCM analysis, the v/c ratio for the critical lanes (right lanes) for the east, west, south and northbound traffic was found to be 0.96, 0.58, 0.62 and 1.41, respectively.

The existing roundabout was first analyzed in VISSIM using its geometric features. All other VISSIM parameters were kept at default. This was done to generate the initial data using the 5 multiple runs. The average delay for the initial five simulation runs are shown in Table 4.15. The descriptive statistics for five runs are shown in Table 4.16.

Runs	**Intersection Delay (s)**
1	19.3
2	19.5
3	18.5
4	27.4
5	11.3

Table 4:15 Existing Model Initial Simulation Delay Data

Intersection Delay (s)	
Mean	19.2
Standard Error	2.55
Median	19.3
Standard Deviation	5.71
Sample Variance	32.56
Kurtosis	1.92
Skewness	0.13
Range	16.1
Minimum	11.3
Maximum	27.4
Sum	96
Count	5
Confidence Level (95.0%)	7.09

Table 4:16 Existing Model Descriptive Statistics

Using Equation 4.1, the minimum required number of runs was also determined to be 17 runs. For reporting the actual result for this research, 17 simulation runs were analyzed in VISSIM to achieve a 95% confidence interval for the existing model.

4.2.1 Existing Double Lane Roundabout Model calibration and Validation

The calibration effort of the existing model in VISSIM begun with the use of default values. This included the gap time, headway, driving behavior, and reduced speed. These values were then adjusted and the result compared to the NCHRP 572 data. The field data used in the NCHRP 572 was missing delay records for the southbound traffic so the travel time data was used to calibrate the existing VISSIM model. For calibration, the headway, reduced speed area, driving behavior and link arrangement were adjusted until the VISSIM travel time was close to the field data. This required several runs (five runs per each calibration effort) in VISSIM. See Appendix C for different calibration trials and validation effort. For the driving behavior, parameters in Table 4.17 were varied within the listed ranges.

The different parameters were varied until there was no further change in the simulation output; this is when additional adjustment resulted in same minimum output error. Table 4.18 shows a comparison of the field travel time data with the result of the final VISSIM trial that gave an acceptable error. For example, W-S indicates a comparison of travel time movement entering from the west leg and exiting to south leg exit where 4.7 seconds was measured in the field and 4.5 seconds was predicted by the final VISSIM model.

Parameters	Data Range
Average Standstill Distance (ft)	(1,3)
Additive Part of Desired Safety Distance	(0, 4)
Multiplicative Part of Desired Safety Distance	(1, 5)
Max Deceleration (Own) (ft/s^2)	(-6, -2)
Accepted Deceleration (Own) (ft/s^2)	(-1.5, -0.5)
-1 ft/s^2 per Distance (Own) (ft)	(50, 150)
Max Deceleration (Trailing) (ft/s^2)	(-5, -1)
Accepted Deceleration (Trailing) (ft/s^2)	(-1.5, -0.5)
-1 ft/s^2 per Distance (Trailing) (ft)	(50, 150)
Minimum Headway (ft)	(0.3, 1)
Safety Distance Reduction Factor	(0, 1)
Max. Deceleration for Cooperative Braking (ft/s^2)	(-5, -1)
Lane Change Distance (ft)	(150, 250)
Emergency Stop Distance (ft)	(3, 7)

Table 4:17 Range of VISSIM Driving Behavior Parameters used in Calibration

Movement	Average Travel Time (s)	
	Field Data	VISSIM Data
W-S	4.7	4.5
W-E	8.7	8.4
W-N	14	13.3
S-N	7.75	11.3
S-W	13.4	13.3
S-S	17.25	17.4
E-W	9.3	9.4
E-S	13.75	12.9
N-S	9.25	6.6
N-E	12.9	10.2

Table 4:18 Field and VISSIM Travel Time Comparison

4.2.2 **Existing Double Lane Roundabout Model Analysis**

After the model was validated, various lane lengths were analyzed following the same procedure as described earlier for the hypothetical model (see Table 3.1). The additional lane length was varied for the same two scenarios as for the hypothetical model after the model was validated. Only the additional lane lengths were varied; all parameters remained the same. For each scenario in the existing model, the letter "E" was used, differentiating these scenarios from the hypothetical model which used "H". Also, for the variation 3 of the existing model, the volume from the north leg was as it represented the leg with the lowest entering volume. As an example, where additional lane lengths at the entry are varied at all four legs, this scenario was represented by represented by EA in this study, where E represents the existing model and A represents all legs.

Under Scenario 1, three variations were considered:

1. Additional lane lengths at the entry at all four legs are varied. This scenario was represented by EA in this study, where E represents the existing model and A represents all legs.

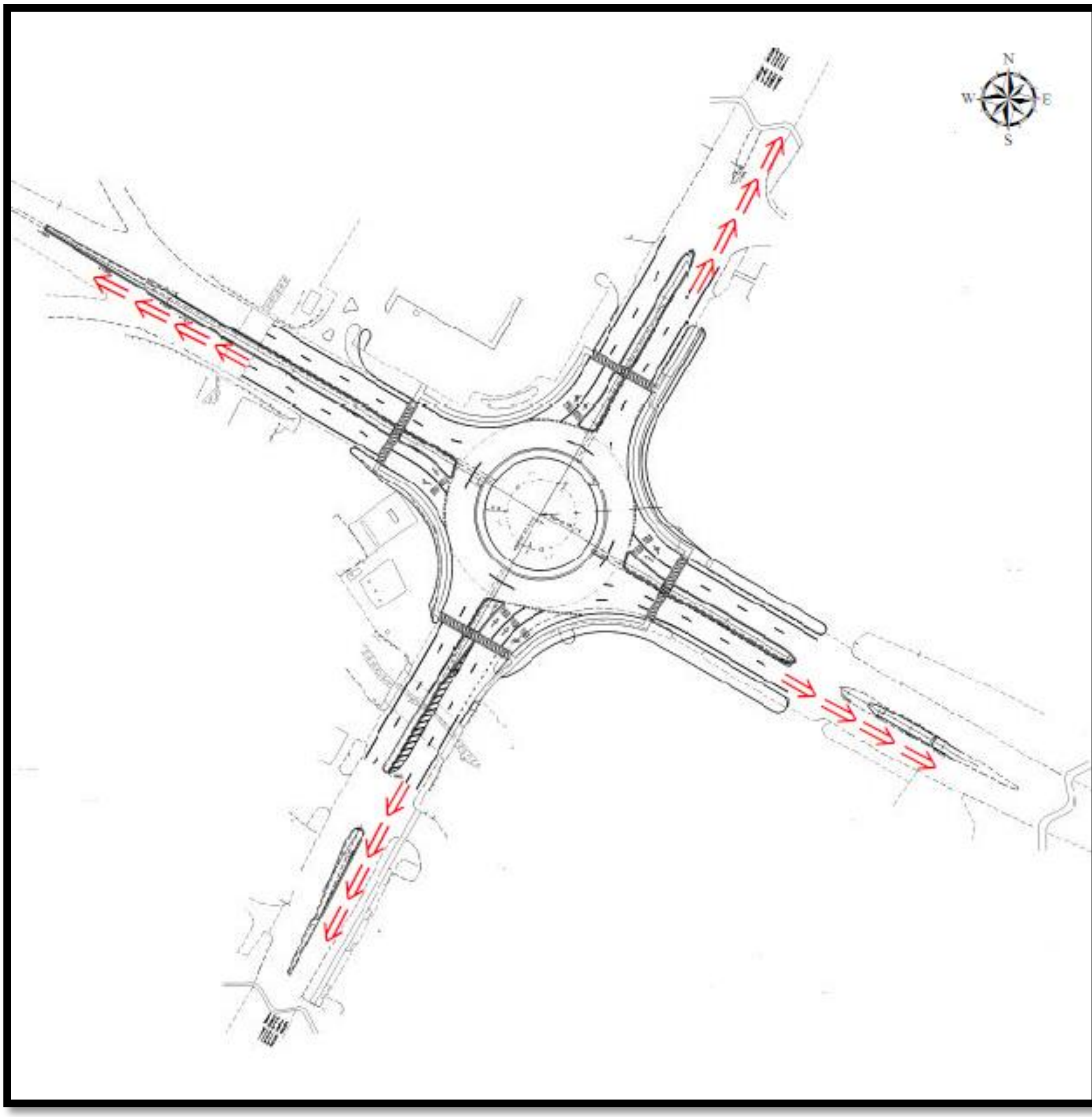

Figure 4:10 EA Scenario Variations

2. An additional lane at the entry with the maximum volume (south leg) is varied. This scenario was represented by ES, where E represents the existing model and S represents south leg.

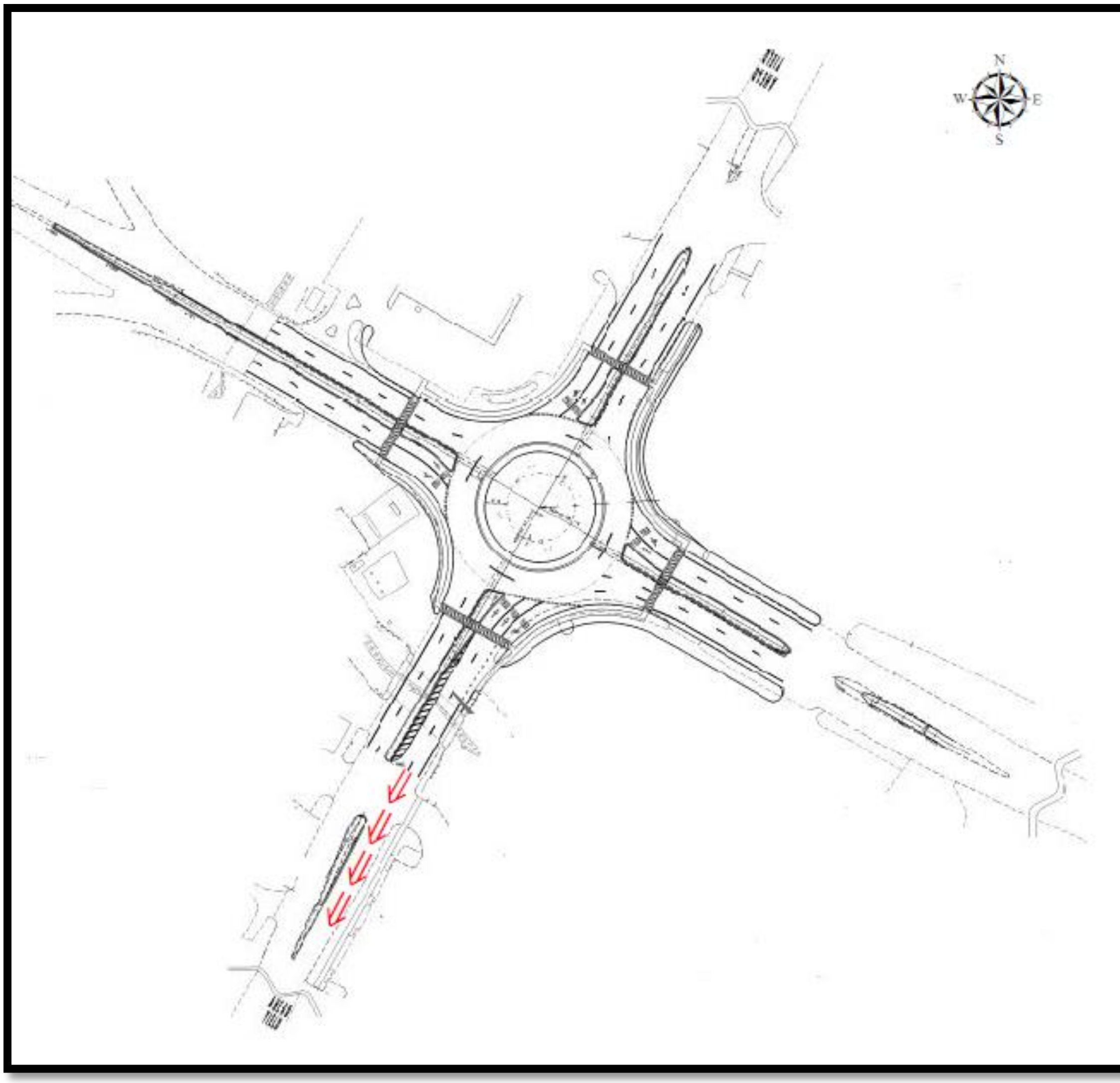

Figure 4:11 ES Scenario Variations

3. An additional lane at the entry with the least volume (north leg) is varied. This scenario was represented by EN, where E represents the existing model and N represents north leg.

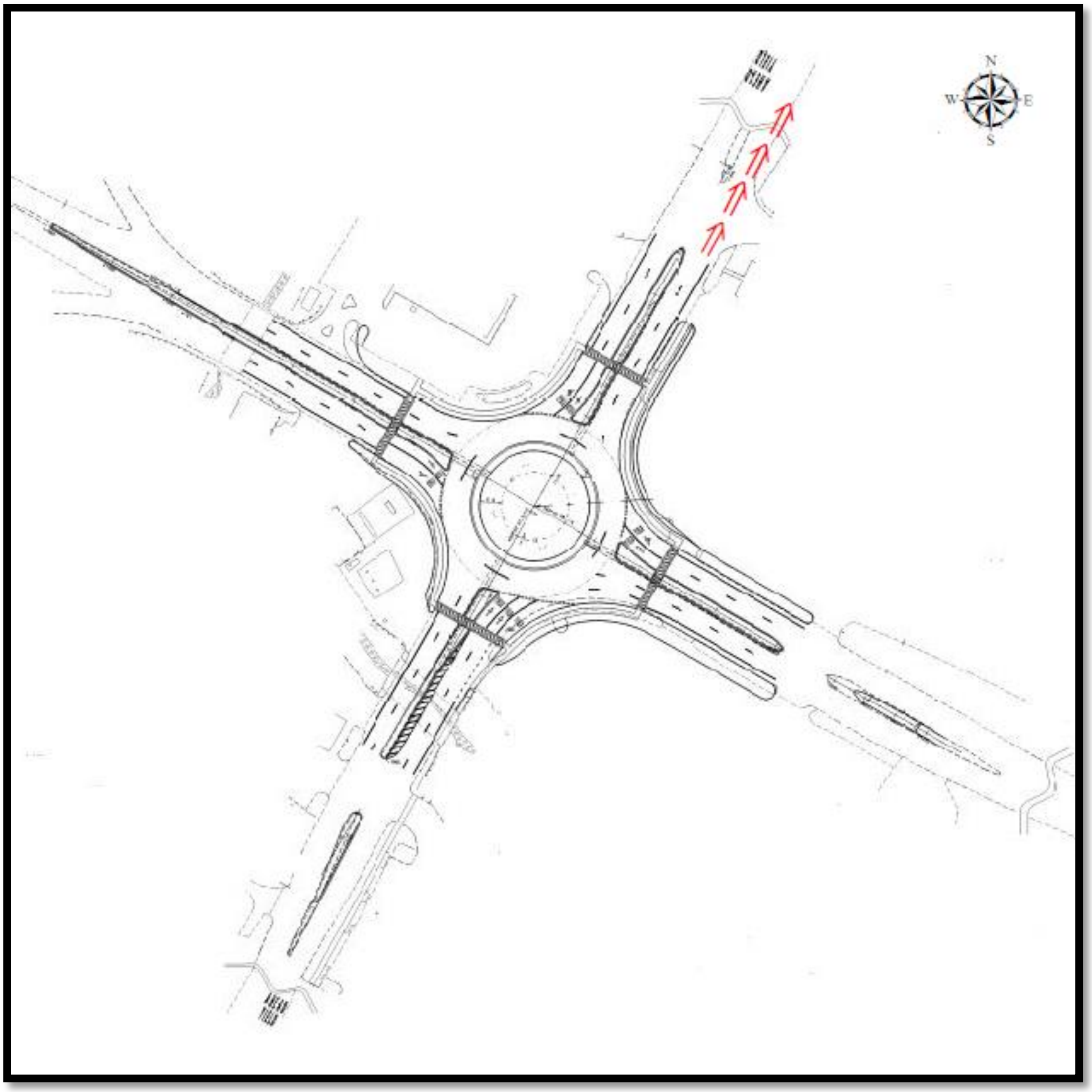

Figure 4:12 EN Scenario Variations

Under Scenario 2, three variations were considered:

1. Additional lane lengths at the entry and exit at all four legs are varied at the same time. This scenario was represented by EAX in this study, where E represents the existing model, A represents all legs and X represents exit.

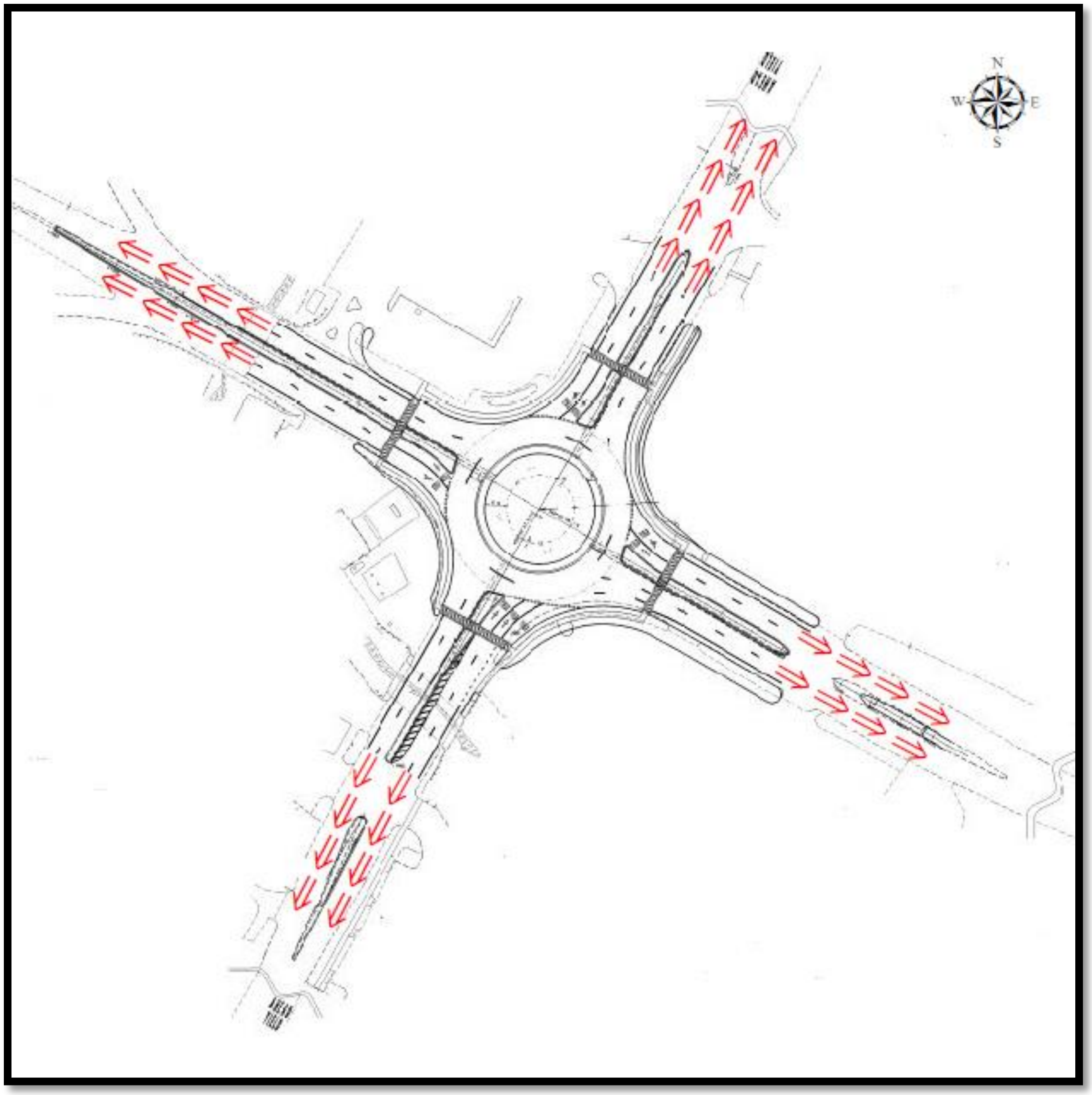

Figure 4:13 EAX Scenario Variations

2. An additional lane at the entry and exit with the maximum volume (south leg) is varied at the same time. This scenario was represented by ESX, where E represents the existing model, S represents south leg and X represents exit.

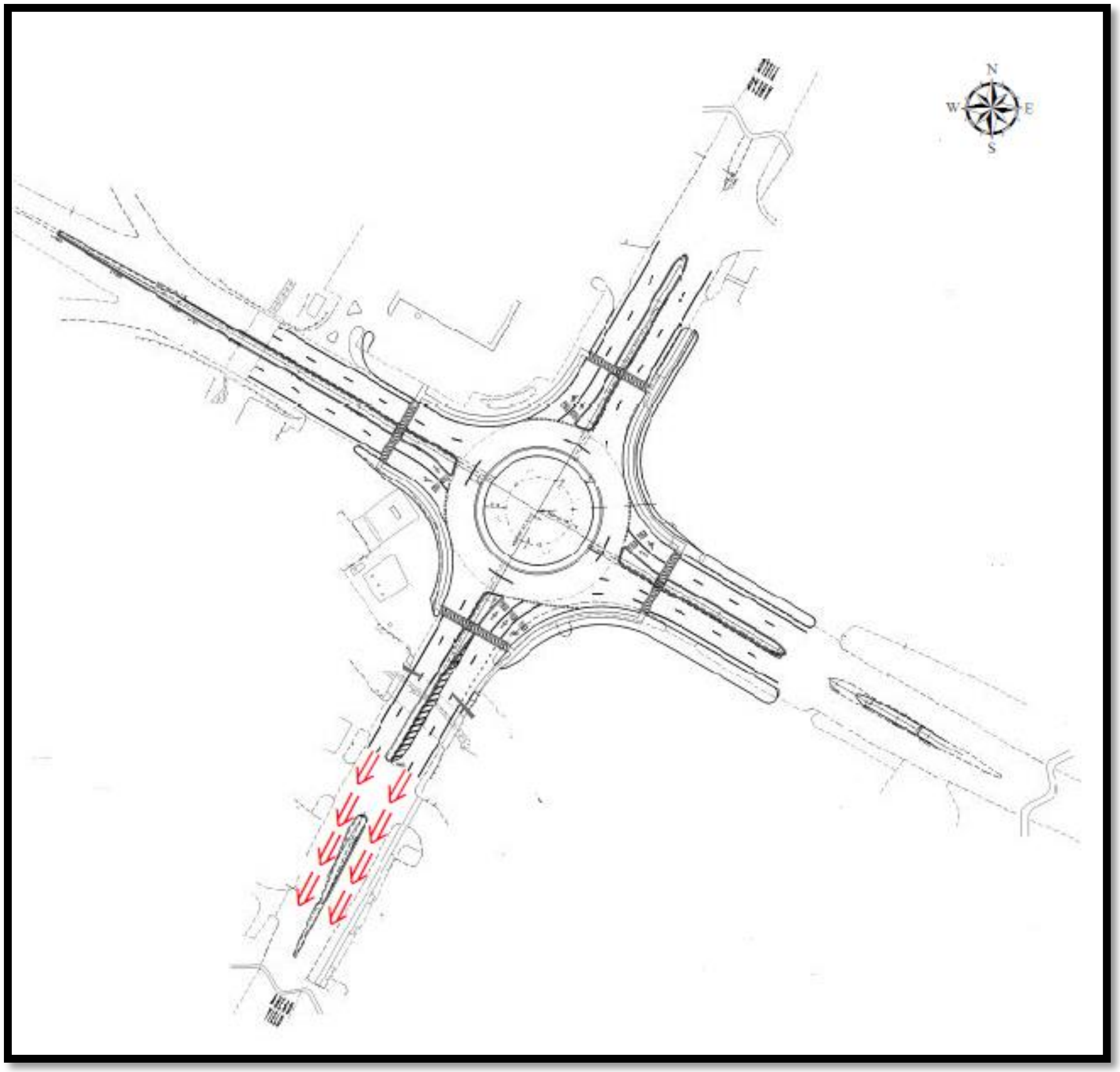

Figure 4:14 ESX Scenario Variations

3. Only one additional lane at the entry and exit with the least volume (north leg) is varied at the same time. This scenario was represented by ENX, where E represents the existing model, N represents north leg and X represents exit.

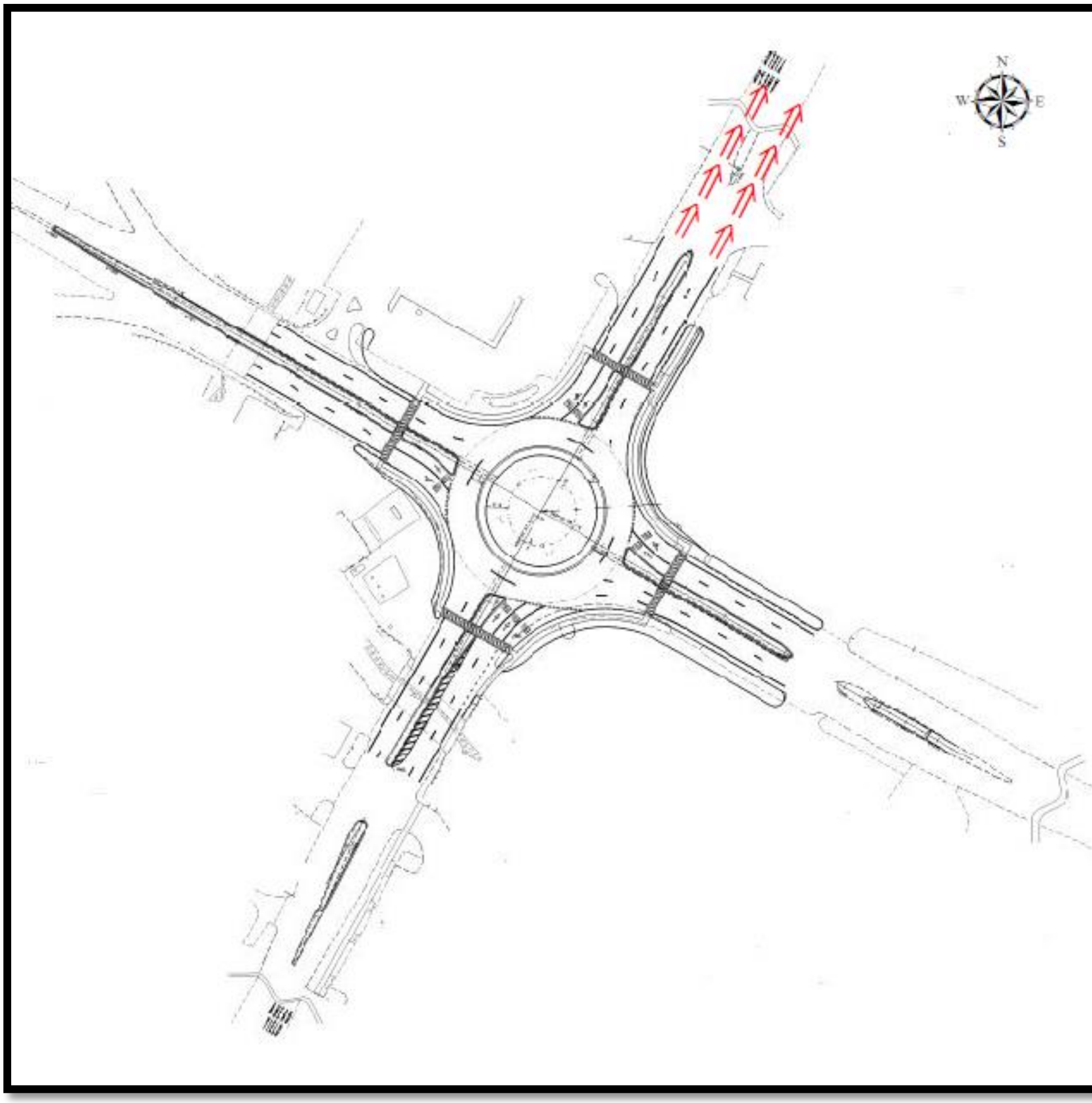

Figure 4:15 ENX Scenario Variations

Since the existing model had varying additional lane lengths of 150 to 180 feet, the following lengths were analyzed for both scenarios: 0 feet, 50 feet, 100 feet and the existing lengths (see Figure 3). Also, 100 feet, 200 feet, 300 feet and 400 feet were added to the exiting additional lane lengths and analyzed in VISSIM to study the effect of longer lengths on roundabout operation. The VISSIM lane closure feature was utilized to make the zero foot length possible.

CHAPTER 5 : RESULTS AND DISCUSSION

The delay and speed data for the hypothetical model is shown in Figure 5:1 through 5:12. The delay data reported is the difference between the measured travel time and free flow travel time from 250 feet approaching the yield line to the exit line on the circulatory roadway. It is the average of the 17 simulation runs. Figure 5 also shows the average speed between these points. Data from the hypothetical model shows that the highest delay point value was when the model had a single lane (zero additional lane length) for all scenarios. There was no significant difference between scenario 1 and 2. The delay data was slightly higher for scenario 2 (when the additional lane length at the entry and exit were varied at the same time). Increasing the length up to approximately 150 feet was effective in reducing delay, but beyond that point there was no significant decrease. In general, an increase in lane length resulted in an increase in vehicle speed.

The analysis of the different variations showed that increasing the length on all four legs at the same time was more effective than just increasing the length on one leg. Increasing the length on the leg with the least volume slightly increased delay at the intersection. As the speed data shows, increasing the lengths caused the speed to increase at the entries; this increases the time at which vehicles reach the circulatory roadway. When more vehicles reach the circulatory roadway within a short period of time the conflicting flow increases and reduces the likelihood of finding an acceptable gap. It is for this reason that the delay increases even though speed will be increasing. Increasing the lengths on just one leg reduced the delay on just that entry, but resulted in allowing more vehicles in the circulatory roadway and increased the conflicting flow for other entries. Increasing the length on the entry with the least volume (minor road) increased the conflicting flow and caused delay on the major road. The delay on the minor road which had minimum effect on the intersection was decreased but the delay on the major road increased. Increasing the length on the entry with the highest volume was more effective than increasing the length on the entry with the lowest volume. This was because the delay on the major road, which affects the entire intersection's delay the most, was reduced. Increasing the length on just one entry (either highest or lowest volume) was not as effective as increasing all four legs at the same time because increasing the length on all four legs reduced the delay on each approach, thereby reducing the delay for the entire intersection.

Wu (2006) suggested balancing the exit and entry capacities in order for the potential of widened entry to be achieved. By balancing the capacities, Wu (2006) suggested that bottleneck effects at the exit can be avoided. From this data, the double lane exit did not affect the delay at the intersection. The difference was more noticeable within short intervals of zero to 150 feet; beyond 150 feet, increasing the exit length did not result in any significant change in the delay. This could be due to the fact that low volumes (or v/c ratios) were considered. It is also possible that the conflict at the exit was minimal because that the roundabout configuration was carefully laid out per NCHRP (2010) guidelines.

The same variations were applied to the existing roundabout in Brattleboro, Vermont. Observations during site visits (Spring 2013) to this roundabout determined that some adjustments to improve its operation during peak hours were needed. During off peak hours, the roundabout operates exceptionally well on all approaches. During peak hours, the south approach sees long queues with associated delays that extend to an average 23 seconds from approximately 250 feet upstream from the yield line. The high traffic in this direction is due to more dense development of restaurants, offices and other businesses south of this roundabout. Under free flow conditions, the travel time from approximately 250 feet upstream to the yield line was measured to be about 7 seconds (but during peak hours, this short interval takes about 30 seconds of travel time). During the peak hour, the east, west and north legs yield increases in delay, while they operate exceptionally well during off peak hours.

In order to evaluate the operations at this roundabout, the length variation applied to the hypothetical model was also applied to the Brattleboro roundabout model in VISSIM after calibration and validation. The results conform to the findings from the hypothetical model. On the east, west, north and south legs of this roundabout, about 180, 160, 150 and 180 feet of respective lane length exist at both entry and exit. The additional lane lengths were decreased so that all lengths were zero (single lane), 50 and 100 feet using the previously stated scenarios. Shorter lengths within 150 feet on all legs resulted in the most significant decrease in delay. Increasing the existing length by 100 foot increments at all legs at the same time resulted in the less change in delay. Zero foot lengths resulted in the highest delay and delay decreased with increasing lengths up to the existing lengths. As noticed in the hypothetical model, adjusting just one leg was not as effective as adjusting all legs at the same time. Adjusting just the leg with the least volume was the least effective means of improving delay. There was no significant difference in varying the length on the exit lane; the difference was more noticeable within short intervals of zero to 150 feet but beyond 150 feet, increasing the exit length did not result any significant change in the delay.

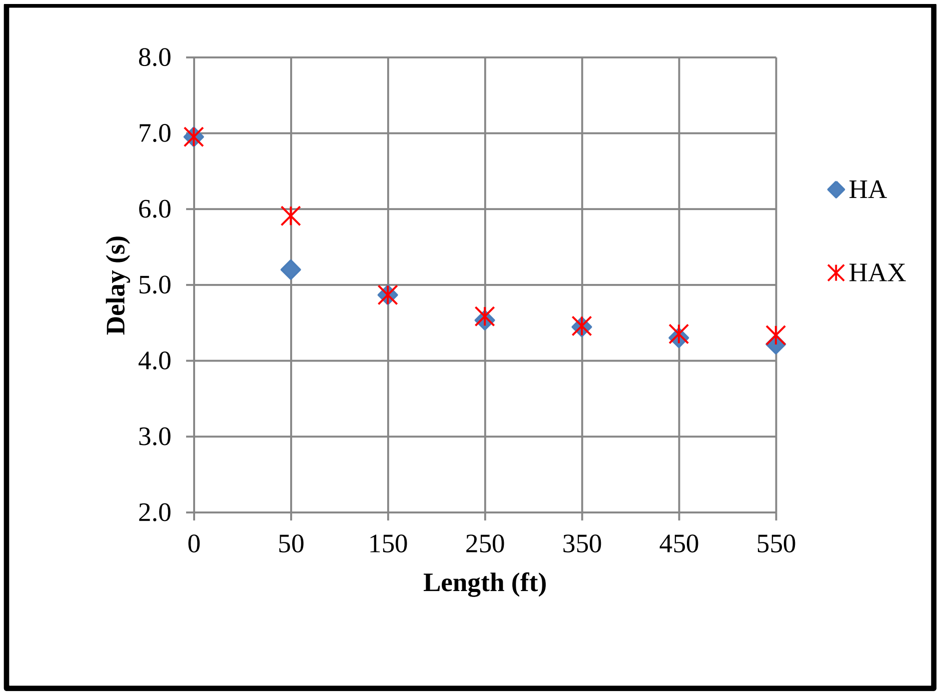

Figure 5:1 Delay Data for (HA) and (HAX) Scenarios

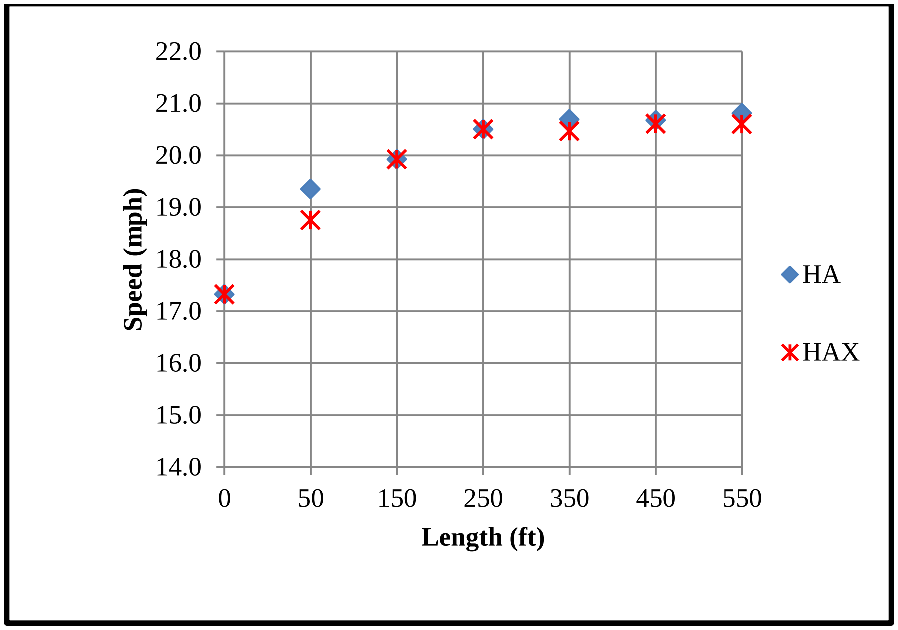

Figure 5:2 Speed Data for (HA) and (HAX) Scenarios

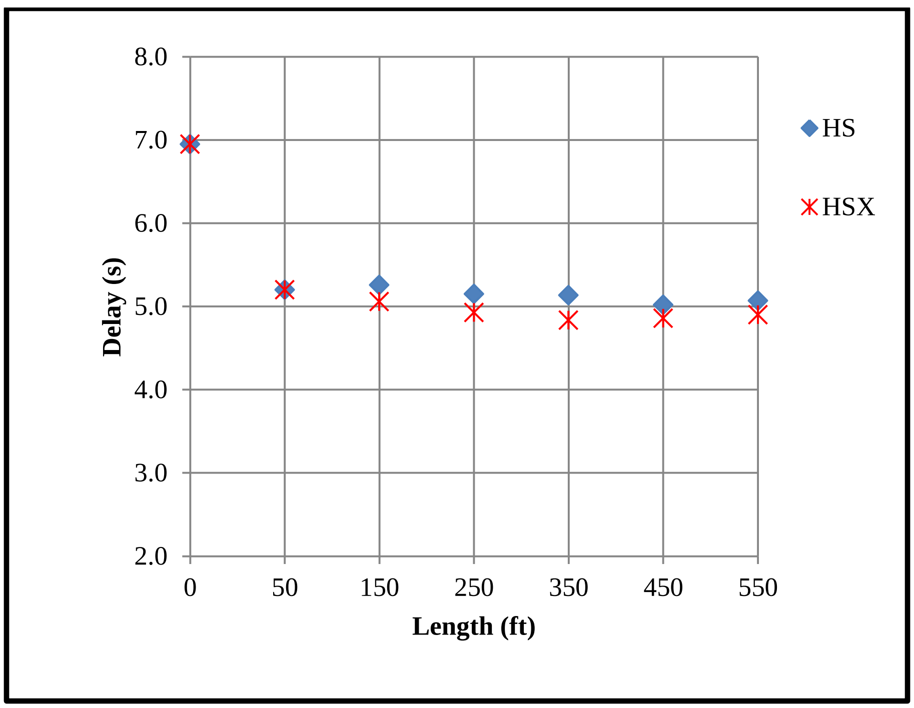

Figure 5:3 Delay Data for (HS) and (HSX) Scenarios

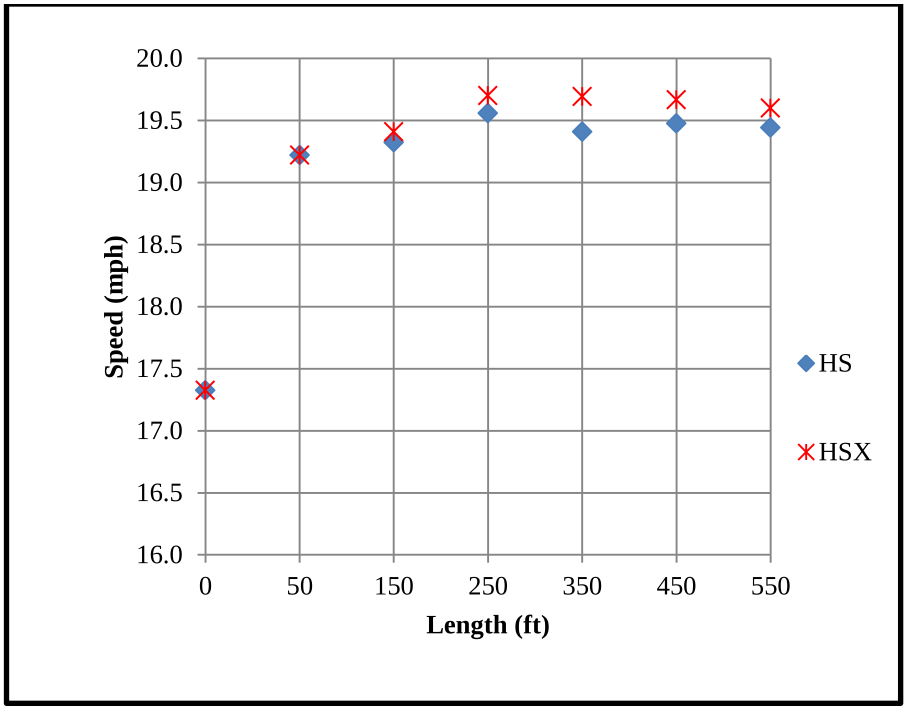

Figure 5:4 Speed Data for (HS) and (HSX) Scenarios

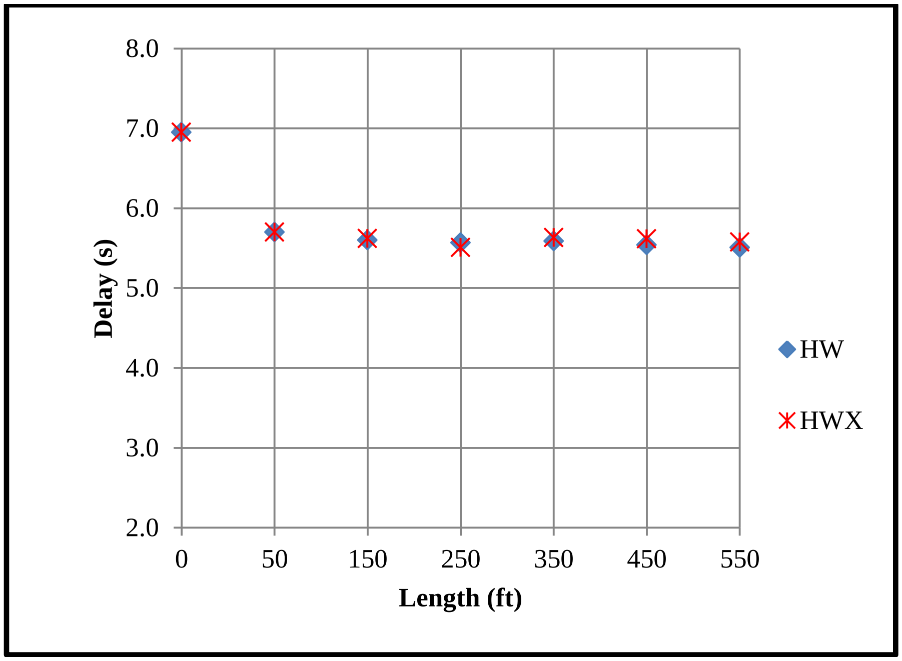

Figure 5:5 Delay Data for (HW) and (HWX) Scenarios

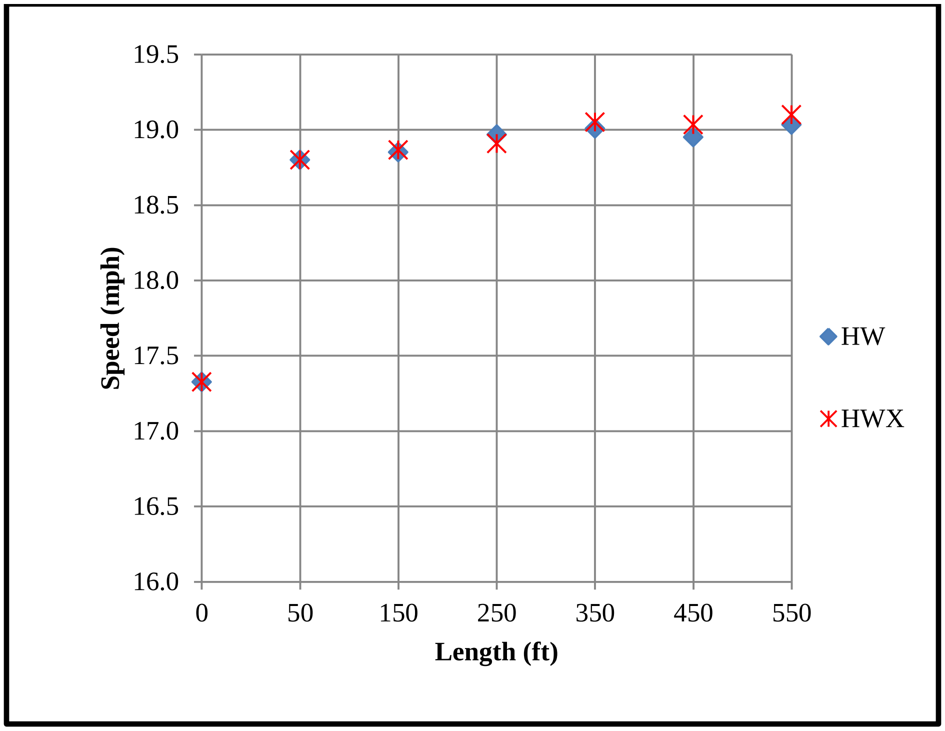

Figure 5:6 Speed Data for (HW) and (HWX) Scenarios

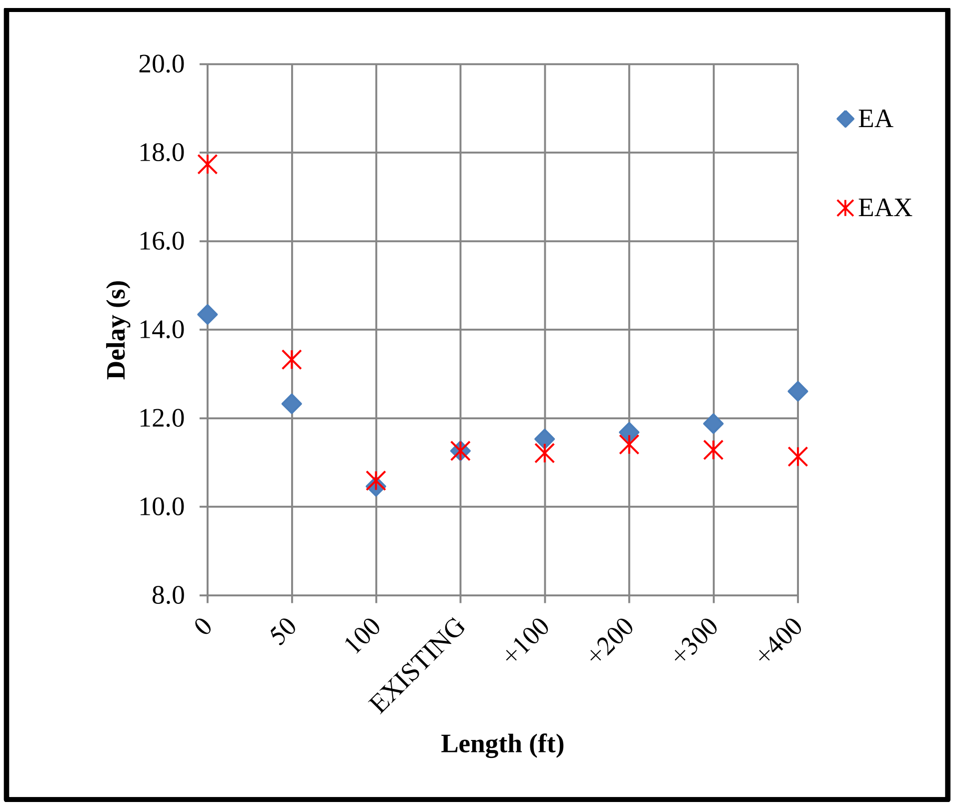

Figure 5:7 Delay Data for (EA) and (EAX) Scenarios

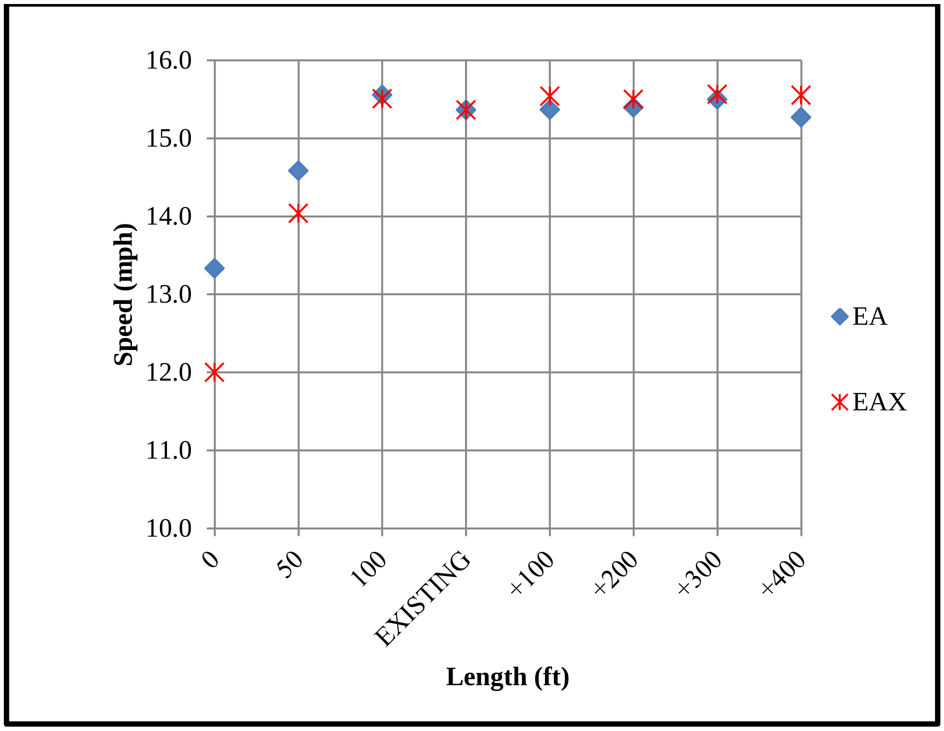

Figure 5:8 Speed Data for (EA) and (EAX) Scenarios

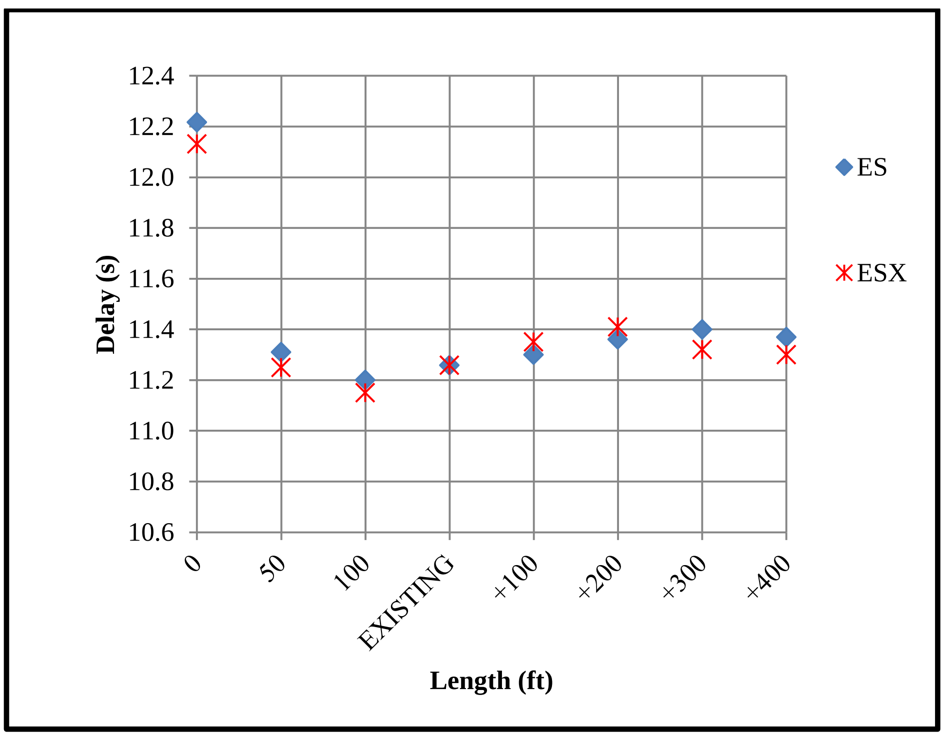

Figure 5:9 Delay Data for (ES) and (ESX) Scenarios

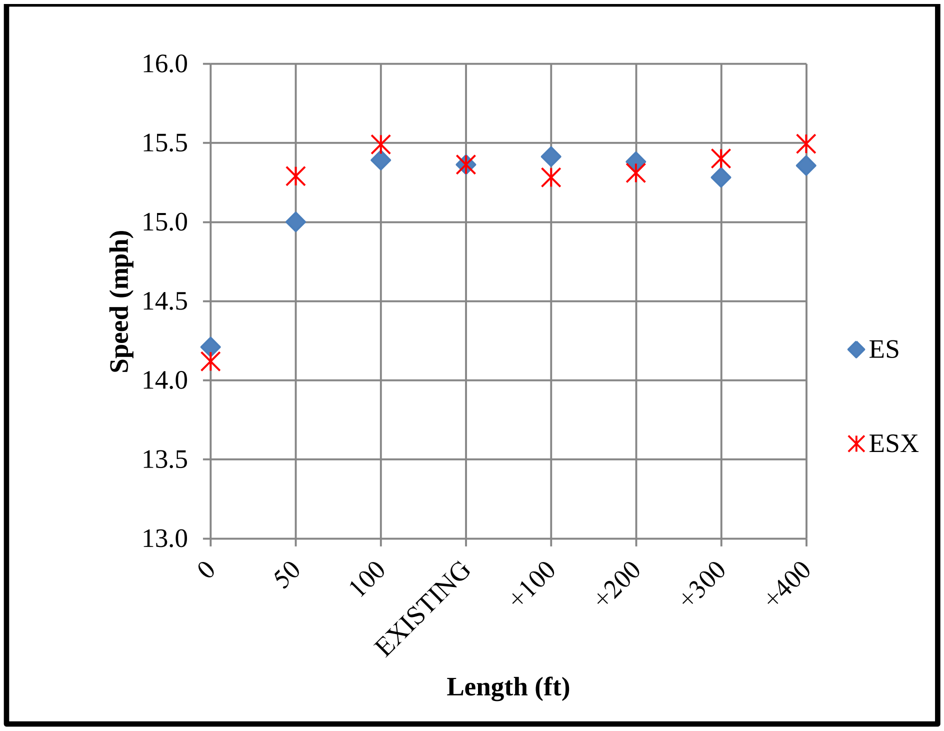

Figure 5:10 Speed Data for (ES) and (ESX) Scenarios

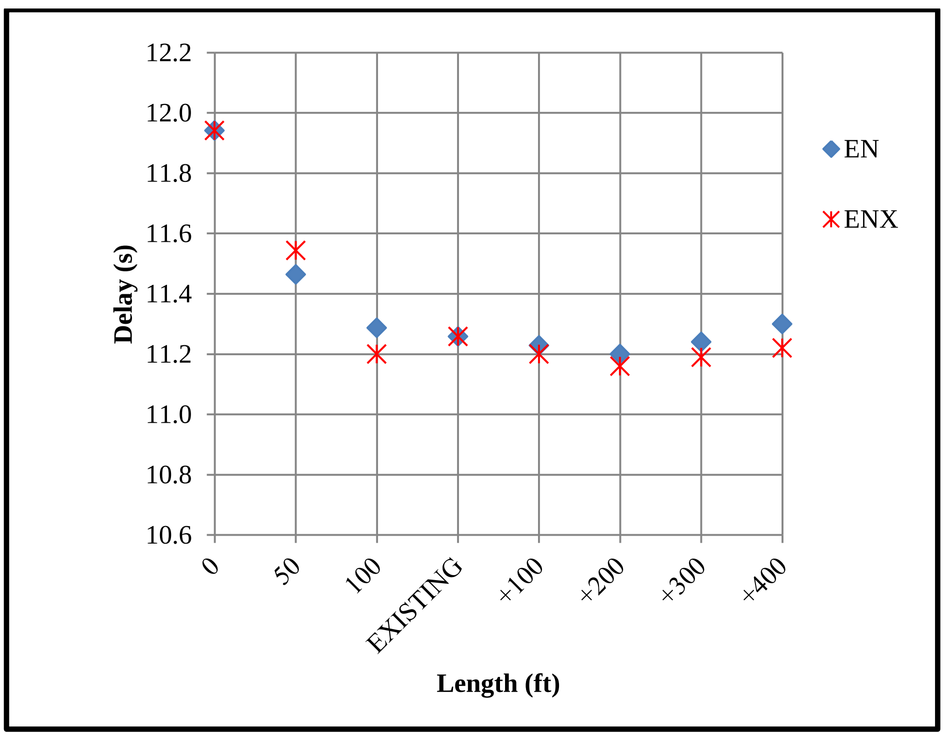

Figure 5:11 Delay Data for (EN) and (ENX) Scenarios

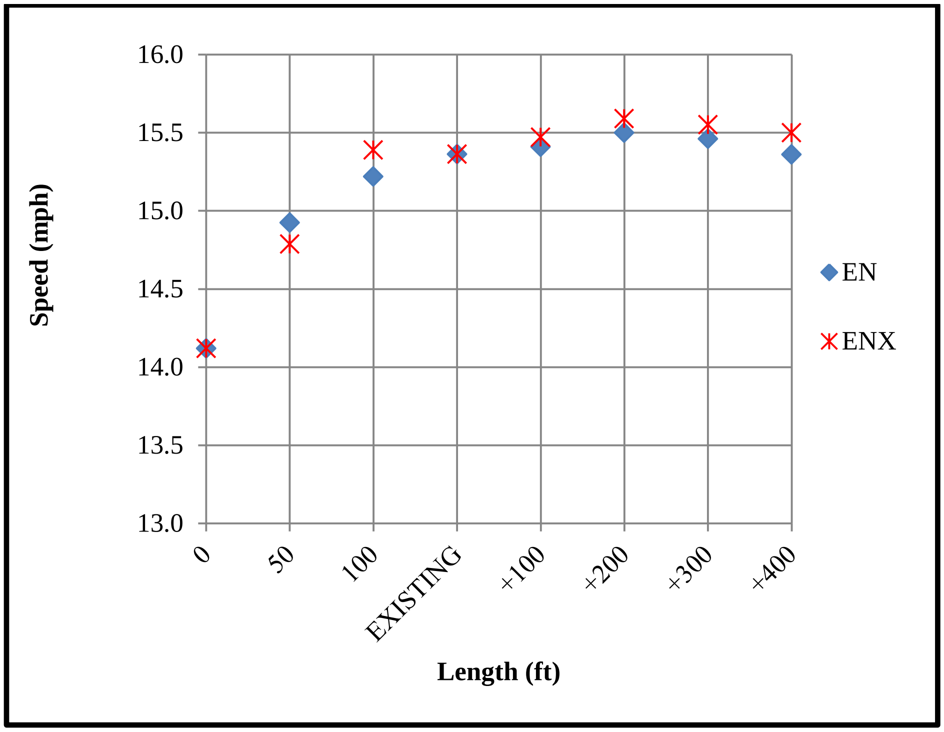

Figure 5:12 Speed Data for (EN) and (ENX) Scenarios

As noted earlier, varying all legs at the same time yielded the best result in delay reduction. To find out the correlation relationship between additional lane length and delay, the plot of the delay versus length for the scenario where all legs were varied was used. In this case the VISSIM delay data for the entire intersection was analyzed. The VISSIM delay results reported earlier looked at delay between specified sections on the roundabout. This helped with the calibration effort as it made it easier to compare the VISSIM data with field data within the same section. In order to establish a relationship between the length and the delay it is appropriate to look at the entire roundabout delay. The entire roundabout delay is the average delay experienced by all vehicles that utilizes the roundabout for the entire length of travel. The plot of this delay versus the length of the additional lane length the hypothetical and existing model models are shown in Figure 5.13 and Figure 5.14 respectively. The delay trend for the entire intersection follows the same trend as the sections specified earlier. The delay decreases sharply to about 150 foot length of additional lane and levels out.

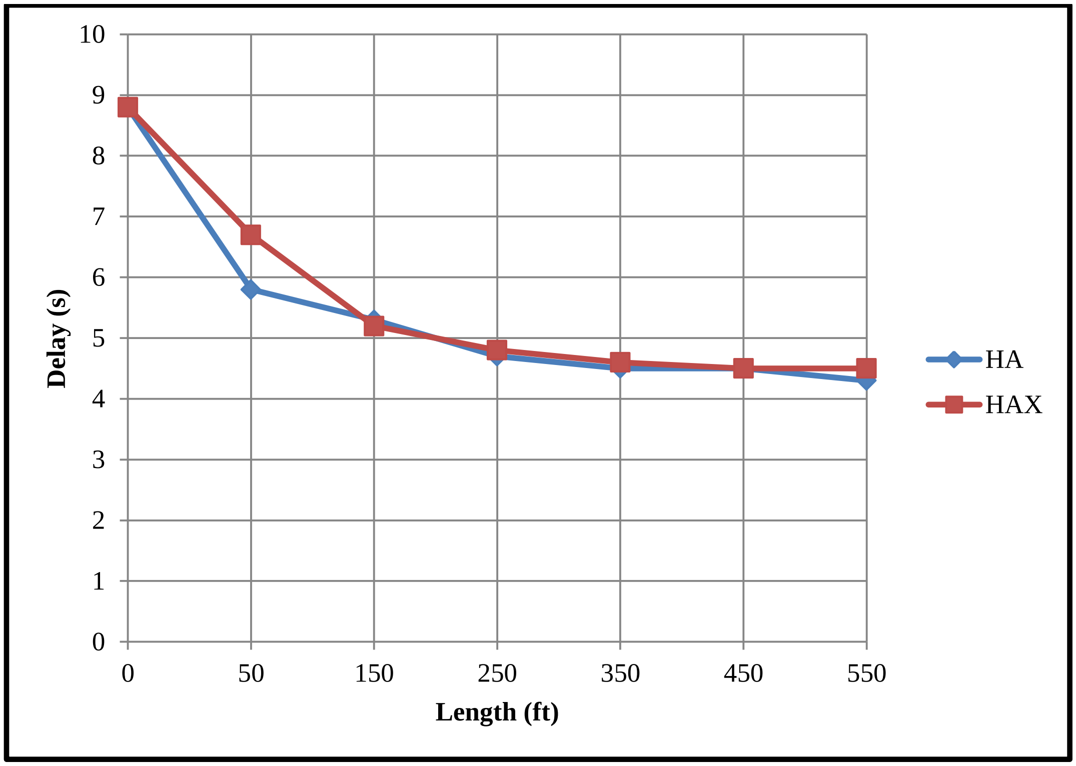

Figure 5:13 Entire Roundabout Intersection Delay Data for Hypothetical Model

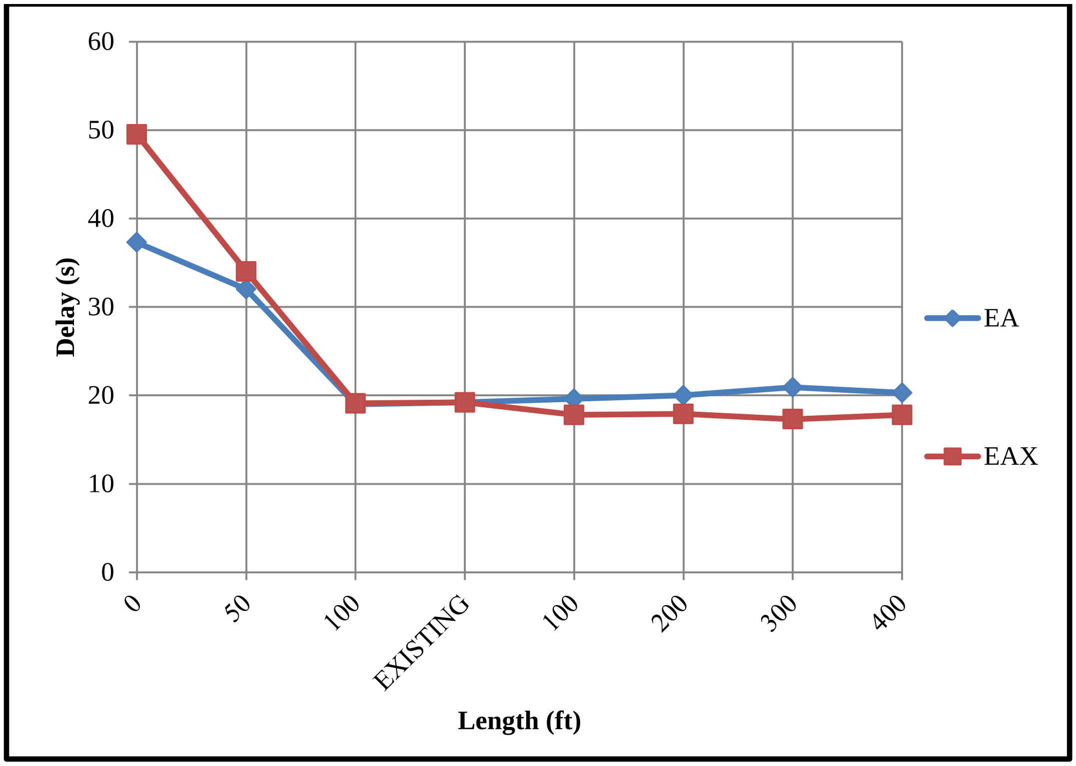

Figure 5:14 Entire Roundabout Intersection Delay Data for Existing Model

To establish a correlation between the delay and the additional lane length, the plot of the existing roundabout data was fitted with a negative exponential based curve for the existing lengths of the existing model. As noted earlier, the longer lengths were not effective in reducing delay so the true correlation laid between zero and about 150 feet. Since the existing roundabout used in this analysis had lengths approximately within this range, the correlation up to the existing lengths was established. Figure 5.15 shows the plot of the delay and versus the length for the scenario where only the entry was varied and the Figure 5.16 shows the scenario where both entry and exit were varied at the same time for all legs.

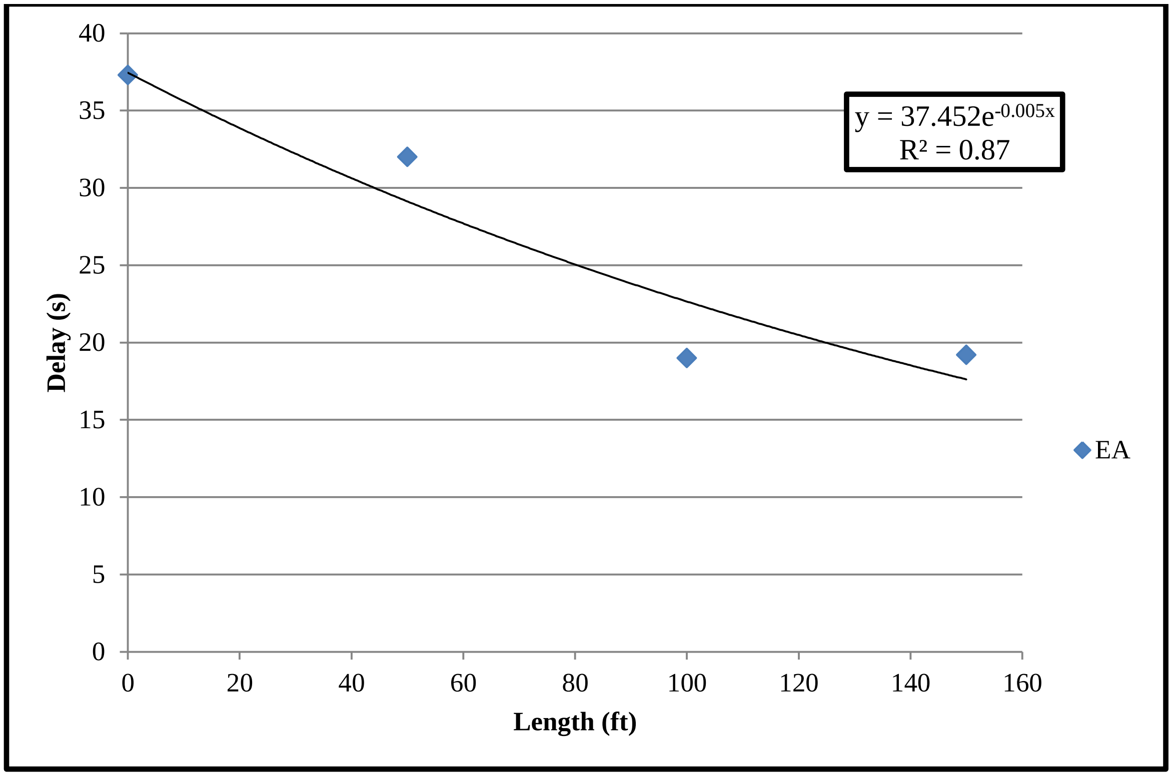

Figure 5:15 Scenario 1 Delay and Additional Lane Length Relationship

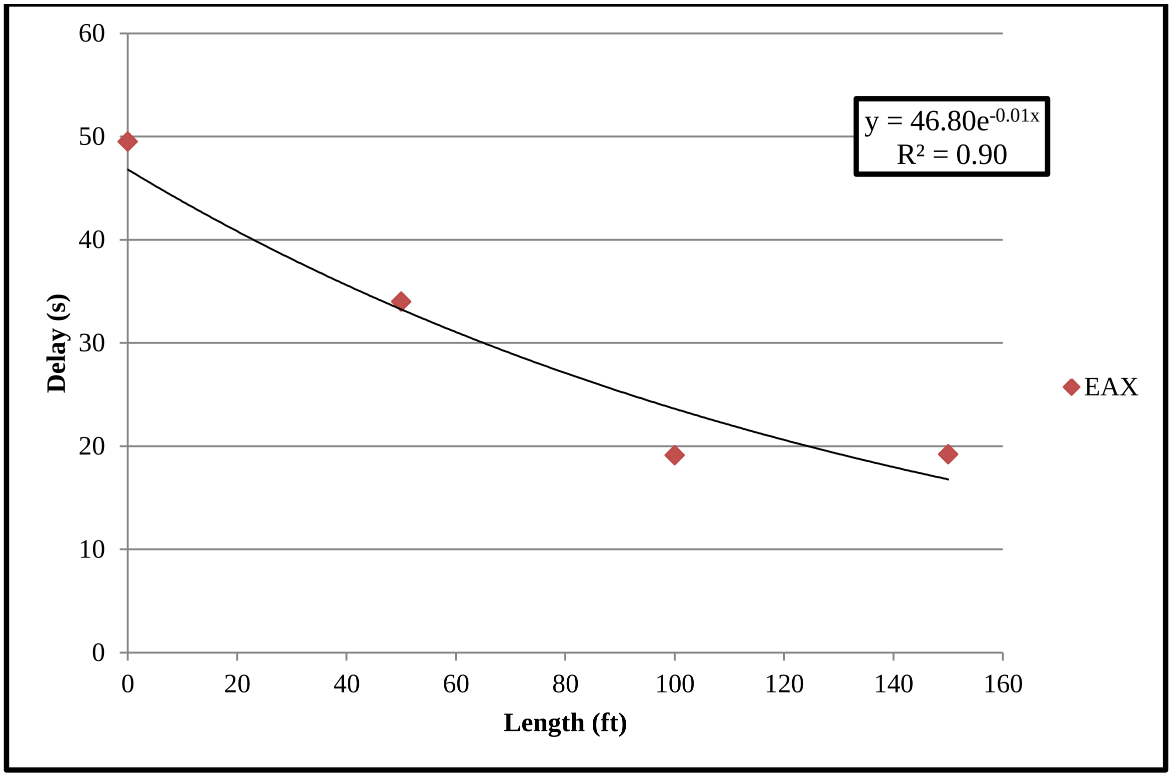

Figure 5:16 Scenario 2 Delay and Additional Lane Length Relationship

The relationship between delay and additional lane length for a roundabout where only the entry lengths are varied for all legs is summarized in Equation 5.1; the R-squared value was 0.87.

$$Delay = 37.452e^{-0.005x} \tag{5.1}$$

where:

x is the length of the shared short lane length.

In this case all lane lengths at the roundabout are assumed to be equal and of shorter lengths approximately between zero and 150 feet. The relationship between delay and shared short lane length for a roundabout where both the entry lengths are varied for all legs is summarized in Equation 5.2; the R-squared value was 0.90.

$$Delay = 46.80e^{-0.01x} \tag{5.2}$$

where:

x is the length of the shared short lane length.

In this case also all lane lengths at the roundabout are assumed to be equal and of shorter lengths approximately between zero and 150 feet. This equation applies to all roundabouts with a degree of saturation less than 0.80 and having approximately equal additional lane lengths within zero and 150 feet. Also for this equation to be applicable, the roundabout should be two lanes with an alignment of 90 degrees or fairly close to 90 degrees to yield similar results. One important observation for the entire intersection delay is the difference between the two scenarios. There was not a significant difference between the delay data for the two scenarios when the delay was analyzed for the sections stated earlier. But when delay for the entire intersection was analyzed, the scenario where both the entry and exit were varied showed much more drastic change in delay than the scenario where only the entry was varied for shorter lengths. This is evident in the slope of the line on a logarithmic scale, 0.251 and 0.342 respectively for EA and EAX scenarios. This means that depending on where the bottleneck results due to increasing only the entry, certain parts of the roundabout may experience reduction in delay.

Based on the results of the analyses presented in this chapter, the significances of the hypotheses of this research are summarized in Table 5.1.

Hypothesis	Statement	Significant
H1	**Shorter additional lane lengths are more effective in reducing delay than longer lengths.**	**Yes**
H2	**Adjusting the additional lane length has to be done concurrently with the exit lane length in order to reduce delay.**	**No**
H3	**Adjusting the additional lane length on all legs is more effective in reducing delay than adjusting just one leg.**	**Yes**

Table 5:1 Summary of Research Hypotheses Results

From roundabout delay data analyzed in VISSIM, hypotheses are supported:

Hypothesis H1: **Shorter additional lane lengths are more effective in reducing delay than longer lengths.** Shorter lengths within zero and 150 feet approximately was more effective in reducing delay than longer lengths. Lengths beyond 150 feet yielded less significant decrease in delay.

For the existing model, the results indicate a significant difference in delay for varying lengths about to about 150 feet (approximate length of exiting roundabout's additional lane length) and no significant change in delay by increasing the additional lane length beyond 150 feet. Table 5:2 and 5:4 show the t-test results where VISSIM delay data set of a particular additional lane length is compared with the VISSIM delay data set of another additional length. Table 5:3 and 5:5 show the percentage differences between VISSIM delay data set of the different lengths used scenario 1. Table 5:2 shows the t-test results comparing the delay data set of the different additional lengths under scenario 1 and Table 5:4 shows t-test results comparing the delay data set of the different additional lengths under scenario 2. Both tables show there was a statistically significant difference between the delay data sets up to the additional lane lengths of the existing roundabout, but when the existing lengths were increased by 100 feet increments, there were no significant difference between the delay data sets. Therefore, we reject the null hypothesis that longer additional lane are more effective in reducing delay than shorter lengths. There will not be any significant decrease in delay by increasing the additional lane length at the existing Brattleboro roundabout.

Length	M	SD	t	p
0 feet	37.28	1.76	5.73	0.00
50 Feet	32.02	1.06		
50 feet	32.02	1.06	7.08	0.00
100 feet	18.82	4.03		
100 feet	18.82	4.03	-0.12	0.91
Existing	19.20	5.71		
Existing	19.20	5.71	-0.09	0.93
Existing + 100 feet	19.56	7.19		
Existing + 100 feet	19.56	7.19	-0.11	0.92
Existing + 200 feet	20.00	5.88		
Existing + 200 feet	20.00	5.88	-0.22	0.83
Existing + 300 feet	20.90	6.95		
Existing + 300 feet	20.90	6.95	-0.46	0.67
Existing + 400 feet	23.12	8.22		

Table 5:2 t-test Results Comparing Delay of Different Lengths Under Scenario 1

Length	% Difference
0 feet 50 Feet	-14.11
50 feet 100 feet	-41.22
100 feet Existing	2.02
Existing Existing + 100 feet	1.88
Existing + 100 feet Existing + 200 feet	2.25
Existing + 200 feet Existing + 300 feet	4.50
Existing + 300 feet Existing + 400 feet	5.26

Table 5:3 Delay Percentage Differences Under Scenario 1

Length	M	SD	t	p
0 feet 50 Feet	49.44 34.00	6.22 2.24	5.22	0.01
50 feet 100 feet	34.00 19.10	2.24 3.50	7.98	0.00
100 feet Existing	19.10 19.20	3.50 5.71	-0.03	0.97
Existing Existing + 100 feet	19.20 17.78	5.71 3.90	0.46	0.66
Existing + 100 feet Existing + 200 feet	17.78 17.84	3.90 3.71	-0.02	0.98
Existing + 200 feet Existing + 300 feet	17.84 17.32	3.71 3.38	0.23	0.82
Existing + 300 feet Existing + 400 feet	17.32 17.74	3.38 4.03	-0.18	0.86

Table 5:4 t-test Results Comparing Delay of Different Lengths Under Scenario 2

Length	% Difference
0 feet 50 Feet	-31.23
50 feet 100 feet	-43.82
100 feet Existing	0.52
Existing Existing + 100 feet	-7.40
Existing + 100 feet Existing + 200 feet	0.34
Existing + 200 feet Existing + 300 feet	-2.91
Existing + 300 feet Existing + 400 feet	2.42

Table 5:5 Delay Percentage Differences Under Scenario 2

Hypothesis H2: **Increasing the additional entry lane length should be done with increasing exit lane length in order to reduce delay.** There was no significant difference between the two scenarios when the delay was analyzed for the sections stated earlier. Both Table 5:2 and Table 5:4 show similar results. Table 5:2, where only the additional lane lengths on the entry were varied, the significant differences occurred through the existing lengths. In a similar fashion, on Table 5:4 where both the additional lane lengths on the entry and exit were varied, the significant differences occurred through the existing lengths. A paired-samples t-test was conducted to compare scenario 1 delay data set with that of scenario 2. The t-test results (Table 5:6) indicate no statistically significant difference between the two data sets except for the conditions where there were zero lengths. As stated earlier, the zero lengths are not practical but were included in this study to understand the length limits. Therefore, we accept the null hypothesis that increasing the additional lane length does not need to be done with increasing exit lane length in order to reduce delay for practical purposes.

Length	M	SD	t	p
0 feet	37.28	1.76	-4.21	0.01
0 Feet	49.44	6.22		
50 feet	32.02	1.06	-1.78	0.12
50 Feet	34.00	2.24		
100 feet	18.82	4.03	-0.12	0.91
100 feet	19.10	3.50		
Existing	19.20	5.71	0.00	1.00
Existing	19.20	5.71		
Existing + 100 feet	19.56	7.19	0.49	0.64
Existing + 100 feet	17.78	3.90		
Existing + 200 feet	20.00	5.88	0.70	0.51
Existing + 200 feet	17.84	3.71		
Existing + 300 feet	20.90	6.95	1.04	0.34
Existing + 300 feet	17.32	3.38		
Existing + 400 feet	23.12	8.22	1.31	0.24
Existing + 400 feet	17.74	4.03		

Table 5:6 t-test Results Comparing Scenario 1 and Scenario 2 Delay Data Set

As stated earlier this comparison was only for the sections of the roundabout that were investigated in this research. The results could vary for sections that cover larger parts of the roundabouts. Increasing the entry and exit at the same time could be more effective in reducing delay for the entire intersection or larger sections but for the sections studied in this research, there were no significant difference between the two scenarios.

Length	% Difference
0 feet 0 Feet	32.62
50 feet 50 Feet	6.18
100 feet 100 feet	1.49
Existing Existing	0.00
Existing + 100 feet Existing + 100 feet	-9.10
Existing + 200 feet Existing + 200 feet	-10.80
Existing + 300 feet Existing + 300 feet	-17.13
Existing + 400 feet Existing + 400 feet	-19.36

Table 5:7 Percentage Differences Between Scenario 1 and Scenario 2 Delay Data Set

Hypothesis H3: **Adjusting the additional lane length on all legs is more effective in reducing delay than adjusting just one leg.** Adjusting all lengths on all legs at the same time yielded the most in delay reduction. The analysis of the different variations showed that increasing the length on all four legs at the same time was more effective than just increasing the length on one leg. The t-test results in Table 5:2, 5:4 and 5:6 show that the most effective means of reducing delay is to use shorter additional lane lengths even in the situations where only few legs have to can be adjusted due to right of way restrictions.

CHAPTER 6 : CONCLUSIONS AND RECOMMENDATIONS

The findings from this study are based on double-lane roundabouts with varying approach geometries and additional lane configurations. The delay values reported in this study were measured from 250 feet from the yield line on the approach and the exit where the vehicles exit the circulatory roadway. Delays upstream before the 250 foot line and beyond the exit line were not recorded. Delays beyond these lines could add to the magnitude of the data reported in this study. Understanding how delay varies within this short interval under the above stated conditions is a better representation of roundabout operation as it was used in the NCHRP Report 572 2007.

Analyses of both the hypothetical and existing roundabout models indicated that very long additional lane lengths were not effective in reducing delay at roundabouts. Shorter lengths of up to 150 feet determined to be were the most effective. This finding corroborates with results from the U.K. Department of Transport Design Manual (U.K. Department of Transport 2007) which recommended shorter flare lengths of about 82 feet to effectively increase capacity. The manual points out that longer flare lengths result in higher speed. Delay reduction was even more effective when both the entry and the exit of short lanes are adjusted at the same time. This ameliorates Wu's (2006) suggestion of balancing the exit and entry capacities. The findings from this study can also be applied to flare designs. Where flaring is used, additional analysis is needed if the flaring does not result in two entry lanes. At entries where two full lanes are used, longer lengths will result in the same effects, namely increased speed and less significant change in delay.

In all cases, delay decreased with increasing lengths, but was most effective with shorter lengths between 50 and 150 feet at both the entry and exit. Varying the lengths was more effective if applied to all legs. In the situation where only one leg can be adjusted, the leg with the most volume should be adjusted and length variation should be within the 50 to 150 foot range. If lengths of 150 feet exist, other modification techniques need be applied as longer lengths will be ineffective in reducing delay. Increasing the additional lane lengths allowed vehicles to use the extra space to reach the roundabout at a faster time thus increasing the speed. But when more vehicles enter the roundabout, the conflicting flow increases and, if there are still sufficient gaps in circulating traffic, more entering vehicles are able to enter at a faster rate, reducing delay. It is important to have enough capacity in the circulatory roadway to receive the entering traffic. The NCHRP (2010) addresses design procedure that balances entry, circulatory and exit flow through lane numbers and arrangements. The shorter lengths help regulate the rate of entry at a slow but constant rate than the longer lengths which can result in an instantaneous increase in circulatory roadway flow with less capacity to handle the flow.

The findings from this will help transportation professionals in dealing with roundabout design and operations. This study confirms that additional lane length can be varied in a manner that effectively reduces delay without wasting money on unnecessary lane construction. This study can also be used during the planning and design stage of a new roundabout in order to determine the appropriate additional lane length without expanding resources on the design and construction of unnecessarily long lengths. Additional analysis is needed to determine the effect of different lengths on safety since this study has shown that increasing the lengths increase speed on the approach to the roundabout. The main goal for modifying the old configuration of roundabouts was to reduce speed and thereby increase safety. If increasing the lengths results in increased speed, this could undermine the operational benefits of a modern roundabout. Therefore, determining an appropriate length allows the ability to identify a minimal additional length to improve operations with an implication of minimizing the increase in speed on the approach.

CHAPTER 7 : FUTURE RESEARCH

There are several topics related to multilane roundabout entry that should be studied further. This study primarily focused on the impact of flare/additional lane length on roundabouts operation. The relationship between the flare/additional lane length and safety needs further investigation. As this research showed, increasing the lengths increased the speed, but there needs to be an established relationship between the lengths and safety since speed increases. Further research that incorporates pedestrians and bicyclists in the study of the impact of flare/additional lane length on roundabouts operation is also recommended. Such additional study will help transportation professional understand the overall safety performance related to flare/additional lane length. More field data collection is recommended to promote more research on multilane roundabouts. Likewise, data collection for roundabouts with shorter lengths and roundabout with longer lengths needs to be performed to aide with future analysis. Such effort will help with the calibration and validation future models.

In order to apply findings from this study to other roundabouts with different degree of saturation, this study needs to be repeated with roundabouts with varying volumes. The volumes on each approach needs to be varied as well. Three possible scenarios can be studied:

Scenario 1: The additional lane lengths at the entry of all legs can be varied and at each length different volumes can be used for the analysis.

Scenario 2: Both entry and exit additional lane length of all legs can be varied and at each length different volumes can be used for the analysis.

Scenario 3: Scenario one and two can be repeated for only one, two and three legs. Such an approach will provide a comprehensive understanding of multilane roundabout operations in terms additional lane lengths.

VISSIM is a great is a powerful microscopic simulation tool for analyzing roundabout operation however it has a very large number of input parameters which makes the model calibration rather difficult and it takes several hours to build a model. VISSIM needs to add and to identify the driver behavior parameters that list new default values for the driving in the additional lane areas. Parameters that relate specifically to roundabouts need to be added to VISSIM for easy and quick analysis in the practical world. Findings from this study such as the delay and speed data for additional lane lengths can be incorporated into driver behavior parameters. This study focused on a degree of saturation less than 0.80 due to the lack of analytical models that effectively model delay during oversaturated conditions. There was no roundabout to compare the model used in this study with during oversaturated conditions. Transportation professionals still find the existing models to be inadequate in delay prediction during real world oversaturated conditions. Models that effectively model delay during oversaturated conditions need to be developed specifically for roundabouts. If such models are developed, this research can be extended to degree of saturation greater than 0.80.

REFERENCES

Al-Omari, B. H., Al-Masaeid, H. R., Al-Shawabkah, Y. S. (2004). ”Development of a Delay Modelfor Roundabout in Jordan.” Journal of Transportation Engineering, American Society of Civil Engineers, February 2004.

Bared, J. G. and Afshar, A. M. (2009). “Planning Capacity Models By Lane for 2-and 3-lane Roundabouts Using Simulation.” Transportation Research Board, National Research Council, Washington, D.C.

FHWA (2000). US Department of Transportation, Federal Highway Administration, “Roundabouts:An Informational Guide.”, Publication No. *FHWA-RD-00-067,* Washington, D.C., June 2000. FHWA Publication Roundabouts: An Informational Guide web site, <http://www.tfhrc.gov/safety/00068.htm>. Accessed March 1, 2013.

Flannery, A., Elefteriadou, L., Koza, P., and Mcfadden, J. (1998). “Safety, delay, and capacity of single-lane roundabouts in the United States.” *Transportation Research Record: Journal of the Transportation Research Board, No. 1646,* Transportation Research Board of the National Academies, Washington, D.C., pp. 63-70.

Fellendorf M. and Vortisch P.(2001). *Validation of the Microscopic Traffic Flow model VISSIM in different Real-World Situations,* 80th Annual Meeting, Transportation Research Board, Washington, D.C., 2001.

Genesee Transportation Council (2007). ”What is a Roundabout?.” Rochester, N.Y. Web site <http://www.gtcmpo.org/programs/resources/roundabouts.htm>. Accessed May12, 2013.

Hagring, O. (2001). “Derivation of capacity equation for roundabout entry with mixed circulating and exiting flows.” *Transportation Research Record: Journal of the Transportation Research Board, No. 1776,* Transportation Research Board of the National Academies, Washington, D.C., pp. 91-99.

HCM (2010). Highway Capacity Manual. Transportation Research Board, National Research Council, Washington, D.C.

HCM (2000). Highway Capacity Manual. Transportation Research Board, National Research Council, Washington, D.C. Kyte, M., Dixon, M., List, G., Flannery, A. and Rodegerdts, L. (2005). "NCRHP 3-65: Data Collection and Extraction ". Proceeding of TRB National Roundabout Conference, Vail, Colorado.

Kittelson & Associates, Inc. (2005). "Kansas Roundabout Guide- A supplement to FHWA's Roundabouts: An Informational Guide." Kansas Department of Transportation, 2005,Website,<http://www.ksdot.org/burtrafficeng/Roundabouts/Roundabout_Guide/KDOTMaster_100803.pdf>. Accessed March 10, 2013.

Kittelson (2010). Kittelson Modern Roundabouts Inventory Activities Web site. <http://roundabouts.kittelson.com/>. Accessed March 10, 2013.

Kimber, R.M. (1980). *The traffic capacity of roundabouts*. TRRL Laboratory Report LR 942. Transport and Road Research Laboratory, Crowthorne, England.

Li, Z., DeAmico, M., Chitturi, M., Bill, A., and Noyce, D. (2013). **"**Calibration of VISSIM Roundabout Model: A Critical Gap and Follow-up Headway." Transportation Research Board, National Research Council, Washington, D.C.

Mereszczak, Y., Dixon, M., Kyte, M., Rodegerdts, L., and Blogg, M. (2006). "Including exiting
vehicles in capacity estimation at single-lane U.S. roundabouts." *Transportation Research Record:Journal of the Transportation Research Board, No. 1988,* Transportation Research Board of the National Academies, Washington, D.C., pp. 23-30.

NCHRP 3-65 (2004). National Cooperative Highway Research Program, Report 3-65: "Applying Roundabouts in the United States." *Transportation Research Board* of the National Academics, Washington, D.C.

NCHRP Report 572 (2007). National Cooperative Highway Research Program, Web-only Document 94: Appendixes to NCHRP Report 572: "Roundabouts in the United States." NCHRP Report 672 (2010). National Cooperative Highway Research Program: "Roundabouts: An Informational Guide." Second Edition. *Transportation Research Board* of the National Academics, Washington, D.C.

PTV VISSIM User’s Manual 5.30 (2010). Planung Transport Verkehr AG, D-76131 Karlsruhe, Germany.

Rodegerdts, L. (2004). NCHRP 3-65: Applying Roundabouts in the United States. Status Report to the Committee on Highway Capacity and Quality of Service July 2004. Web site at <http://people.sunyit.edu/~lhmi/ahb40/meetings/200406/4771%20HCQS%202004-06.pdf>.Accessed March 1, 2013.

U.K. Department of Transport (2007). Design Manual for Roads and Bridges, Volume 6, Section 2, Part 3.

Trueblood, M. and Dale, J. (2007). “Simulating Roundabouts with VISSIM”. National Roundabouts Conference: 2005 Proceedings. TRB Transportation Research Circular E-C083.

Wu, N. (1997). Capacity of shared/short lanes at unsignalized intersections. In *Proc., Third International Symposium on Intersections without Traffic Signals* (M. Kyte, ed.), Portland Oregon, U.S.A., University of Idaho.

Wu, N. (2006). “Capacity Enhancement and Limitation at Roundabouts with Double-Lane or Flare Entries.” Proceeding of the 5th International Symposium on Highway Capacity and Quality of Service. Volume 2- Technical Papers. Yokohama, Japan

Wan, B., and Rouphail, N. M. (2004). “Using Arena for Simulation of Pedestrian Crossing in Roundabout Areas.” Transportation Research Record: Journal of the Transportation Research Board, No. 1878, Transportation Research Board of the National Academies, Washington, D.C., pp. 58-65.

Wiedemann R. and Reiter, U (1991). “*Microscopic Traffic Simulation: The Simulation System Mission*, “ PTV America, Inc. Web site at ptvag.com/download/traffic/library/Wiedemann.pdf Accessed March 1, 2013.

APPENDIX

Appendix A: Brattleboro Roundabout Field Data

The field data collection effort included video recordings of traffic flow then extracting five events from the recorded videos using keystroke recording software. The keystroke recording software generated a time stamp file whenever any of these five events occurred. These events are listed and described in Table A:1. The event locations have been illustrated on a picture of the south approach of the Brattleboro roundabout (Figure A:1).

Events	Keystroke	Description
Entry time	2	The entry of a vehicle into the roundabout from the approach. The time was recorded when the vehicle crossed the yield line; the lane placement of the vehicle (either left lane or right lane) was recorded for two lane roundabouts. The vehicle type was also recorded.
First-in-queue time	1	The arrival of a vehicle into the server or first in line position on the approach. The time was recorded when the vehicle was about to enter the roundabout (if it did not stop) or the time that it stopped at or near the yield line waiting to enter the roundabout.
Upstream time	z	The passage of a vehicle past a point upstream of the entry point that defines the beginning of the travel time trap.
Conflict time	s	The passage of a vehicle through the conflict point on the roundabout, a point that is adjacent to the point of entry for a minor street vehicle.
Exit Time	a	The exiting of a vehicle from the roundabout.

Table A:1 Events Description

Figure A:1 Event Locations on South Approach of Brattleboro Roundabout

From the time stamp files, the flow rate, delay, travel time, gap times were computed. Table A:2, TableA:3, and Table A:4 show the gap time and delay data extracted from the time stamp files from the Brattleboro roundabout. The field data file was missing gap time and delay data for the north approach. The only file that had data for all four legs was the travel time data file. The travel time data for all four legs is shown in Table A:5. Figure A:2 shows a plotted cumulative distribution for the rejected and accepted gap time using the captured field data. Figure A:3, Figure A:4 and Figure A:5 show the actual field delay and the computed delay for a specified field recording period. In all tables and figures, the right lane and left lane are represented by RL and LL respectively.

Time	FirstQTime				EntryTime				ExitTime		Upstream event			
	Gap data 1		Delay data 1		Gap data 2		Delay data 2		Gap data a	Gap data e	Delay Data z		Average Delay	
	RL	LL	RL	LL	RL	LL	RL	LL			RL	LL	RL	LL
12:37:00 AM	2	1	2	1	1	0	1	0	0	3	5	1	0.1	
12:38:00 AM	10	2	10	2	11	3	11	3	0	11	9	2	12.3	9.1
12:39:00 AM	8	3	8	3	7	3	7	3	1	9	9	3	3.7	2.3
12:40:00 AM	10	1	10	1	10	1	10	1	0	8	10	3	16.9	9.5
12:41:00 AM	8	7	8	7	8	7	8	7	1	16	7	5	16.1	12.2
12:42:00 AM	12	2	12	2	13	2	13	2	0	10	12	3	3.4	3.6
12:43:00 AM	11	2	11	2	10	2	10	2	0	12	13	1	9.9	2.0
12:44:00 AM	10	3	10	3	11	2	11	2	1	9	10	6	17.0	3.1
12:45:00 AM	13	8	13	8	12	8	12	8	0	14	11	6	16.2	22.6
12:46:00 AM	6	6	6	5	7	6	7	6	2	13	7	4	9.2	11.1
12:47:00 AM	7	3	7	4	7	3	7	3	2	18	8	2	9.4	3.7
12:48:00 AM	11	4	11	4	10	5	10	5	1	9	12	4	10.5	3.5
12:49:00 AM	17	2	17	2	17	2	17	2	0	17	15	2	4.0	0.2
12:50:00 AM	13	4	13	4	14	4	14	4	0	12	13	4	4.0	0.2
12:51:00 AM	1	0	1	0	1	0	1	0	0	1	0	0	0.0	

Table A: 2 Flow and Delay Data for South Approach

Time	FirstQTime				EntryTime				ExitTime		Upstream event			
	Gap data 1		Delay data 1		Gap data 2		Delay data 2		Gap data a	Gap data e	Delay Data z		Average Delay	
	RL	LL	RL	LL	RL	LL	RL	LL			RL	LL	RL	LL
12:20:00 AM	1	0	1	0	1	0	3	0	0	4	1	3	2.5	0.0
12:21:00 AM	4	7	4	7	4	6	2	6	0	13	3	5	6.8	4.6
12:22:00 AM	3	4	3	4	3	5	5	5	0	10	4	6	23.1	18.5
12:23:00 AM	3	6	3	6	2	6	4	6	1	10	4	5	5.3	23.2
12:24:00 AM	5	5	5	5	5	5	5	5	0	8	4	4	28.2	9.1
12:25:00 AM	4	3	4	3	4	2	7	2	0	16	3	5	15.9	7.4
12:26:00 AM	5	4	5	4	5	5	5	5	1	11	7	2	8.8	21.5
12:27:00 AM	6	1	6	2	7	1	10	1	0	11	5	3	15.6	19.4
12:28:00 AM	5	6	5	5	5	6	3	6	0	7	3	6	10.4	6.1
12:29:00 AM	10	3	10	3	10	3	7	3	0	10	9	4	3.3	3.5
12:30:00 AM	3	3	3	3	3	3	4	3	0	9	4	3	1.5	3.6
12:31:00 AM	7	4	7	4	7	3	5	3	1	10	8	5	1.8	0.2
12:32:00 AM	4	5	4	5	4	5	2	5	0	9	0	8	25.5	16.3
12:33:00 AM	5	5	5	5	5	6	7	6	0	9	5	3	12.8	23.4
12:34:00 AM	3	3	3	3	2	2	1	2	1	6	8	5	14.2	7.2
12:35:00 AM	6	7	6	7	7	7	0	7	1	12	3	3	26.6	19.3
12:36:00 AM	1	1	1	1	1	2	0	2	1	4	0	1	8.5	9.7

Table A:3 Flow and Delay Data for East Approach

Time	FirstQTime				EntryTime				ExitTime	Upstream event			
	Gap data 1		Delay data 1		Gap data 2		Delay data 2		Gap data e	Delay Data z		Average Delay	
	RL	LL	RL	LL	RL	LL	RL	LL		RL	LL	RL	LL
12:03:00 AM	6	1	6	1	5	0	5	0	7	7	1	0.8	
12:04:00 AM	9	0	9	0	10	1	10	1	7	9	0	7.7	15.1
12:05:00 AM	9	1	9	1	9	1	9	1	14	8	1	6.6	4.4
12:06:00 AM	6	1	6	1	6	1	6	1	13	6	1	6.4	21.6
12:07:00 AM	8	1	8	1	8	1	8	1	10	8	1	8.7	4.9
12:08:00 AM	8	0	8	0	8	0	8	0	8	11	0	2.7	
12:09:00 AM	14	0	14	0	14	0	14	0	10	13	0	1.8	
12:10:00 AM	8	2	8	2	8	2	8	2	5	10	1	3.0	0.4
12:11:00 AM	12	0	12	0	12	0	12	0	13	9	0	2.8	
12:12:00 AM	9	1	9	1	9	1	9	1	11	12	1	0.7	0.6
12:13:00 AM	14	2	14	2	14	2	14	2	11	13	2	4.9	4.9
12:14:00 AM	14	2	14	2	14	2	14	2	6	12	2	4.4	0.0
12:15:00 AM	8	2	8	2	8	2	8	2	7	9	3	6.7	5.1
12:16:00 AM	14	2	14	2	13	2	13	2	13	11	3	3.0	0.1
12:17:00 AM	5	2	5	2	6	1	6	1	8	6	2	6.2	0.0
12:18:00 AM	11	0	12	0	11	1	11	1	12	13	0	12.1	19.6
12:19:00 AM	10	4	9	4	10	4	10	4	8	9	3	7.4	8.1
12:20:00 AM	6	1	6	1	5	1	5	1	7	6	1	13.8	21.0
12:21:00 AM	2	0	2	0	3	0	3	0	1	1	0	7.2	

Table A:4 Flow and Delay Data for West Approach

Approach Location		Movement							
		U-turn		Right		Left		Through	
		RL	LL	RL	LL	RL	LL	RL	LL
Right Lane	North			00:03.8		00:12.9		00:08.9	
	South	00:16.7		00:03.0		00:13.3	00:13.2	00:07.9	
	East			00:02.8		00:13.7		00:08.6	
	West	00:30.7		00:03.6		00:14.7	00:03.6	00:08.6	
Left Lane	North	00:18.7				00:12.8	00:12.9	00:09.4	00:09.8
	South	00:17.8				00:13.0	00:14.0	00:07.5	00:07.7
	East					00:13.8	00:20.2	00:11.4	00:08.6
	West	00:19.3			00:05.8	00:13.5	00:13.3	00:09.1	00:08.4

Table A:5 Average Travel Time Summary

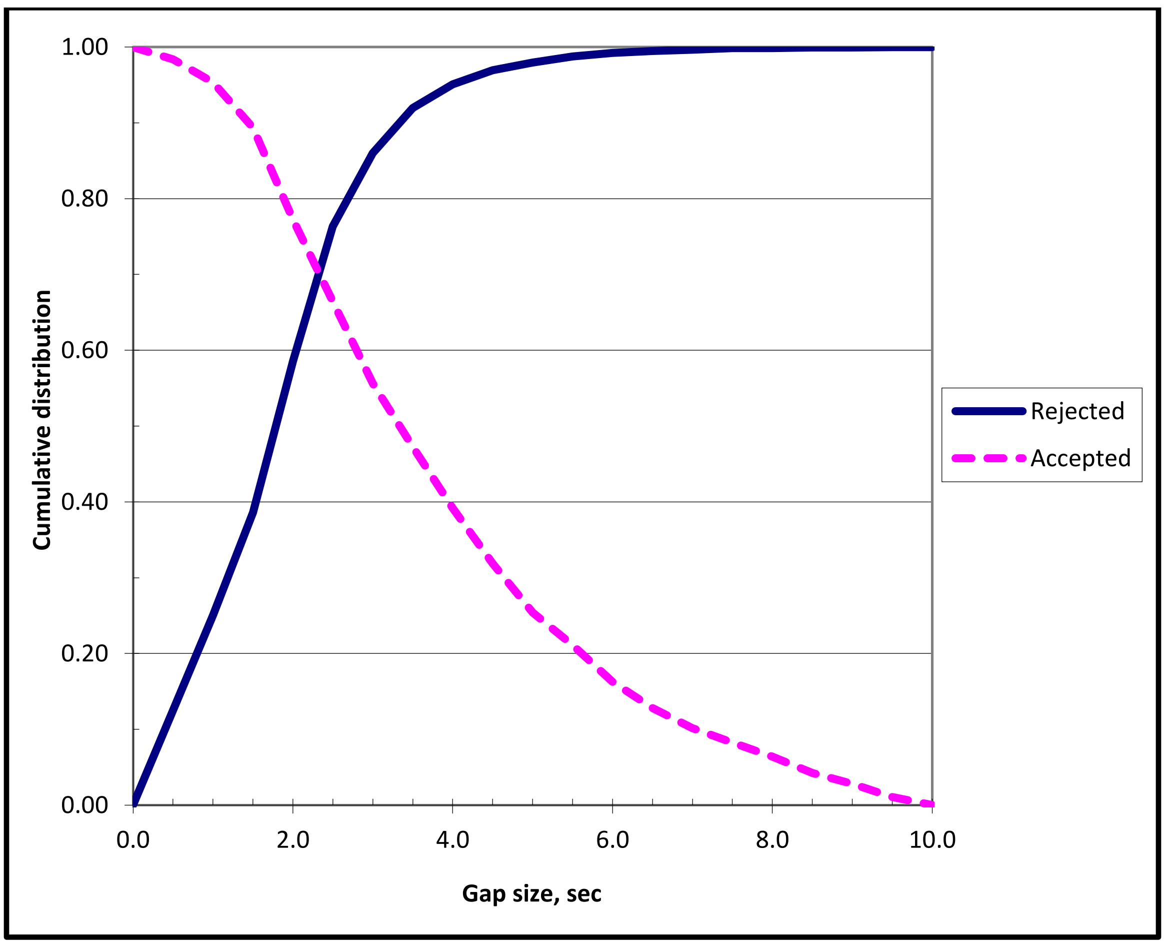

Figure A:2 Entire Intersection Gap Distribution

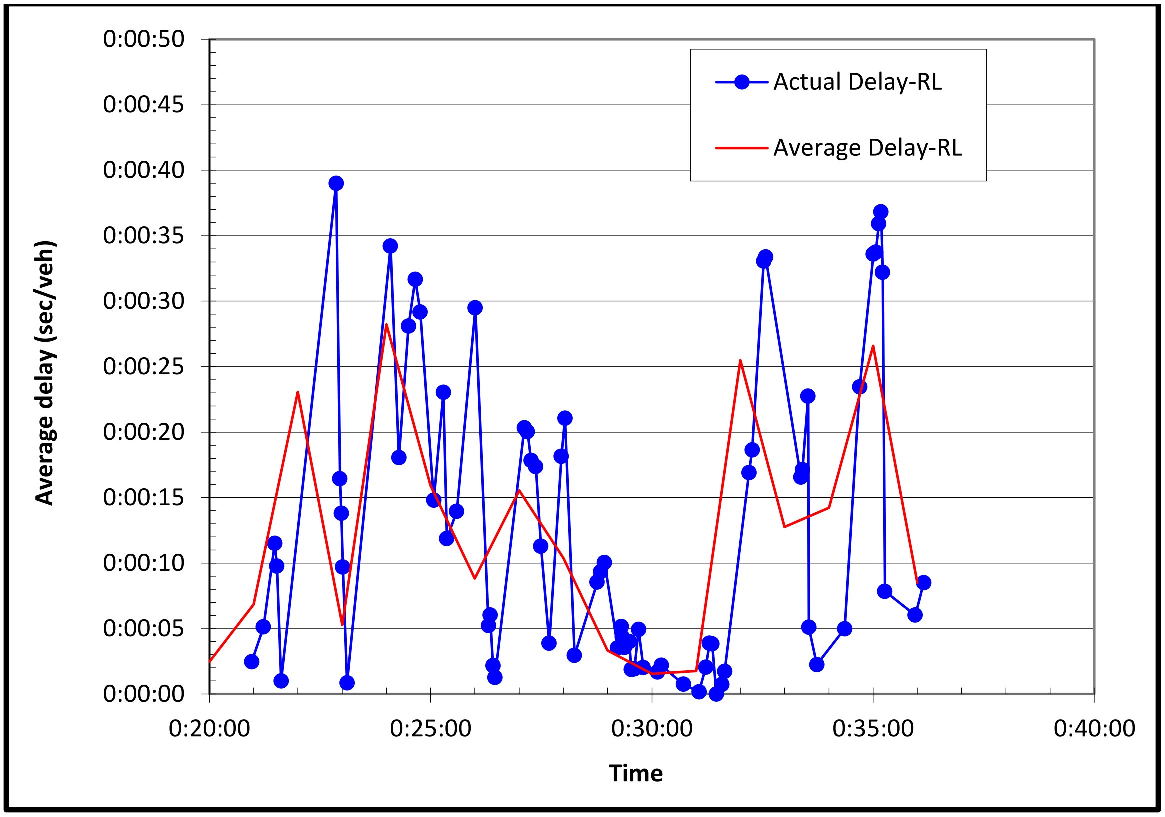

Figure A:3 East Approach Average Delay Chart

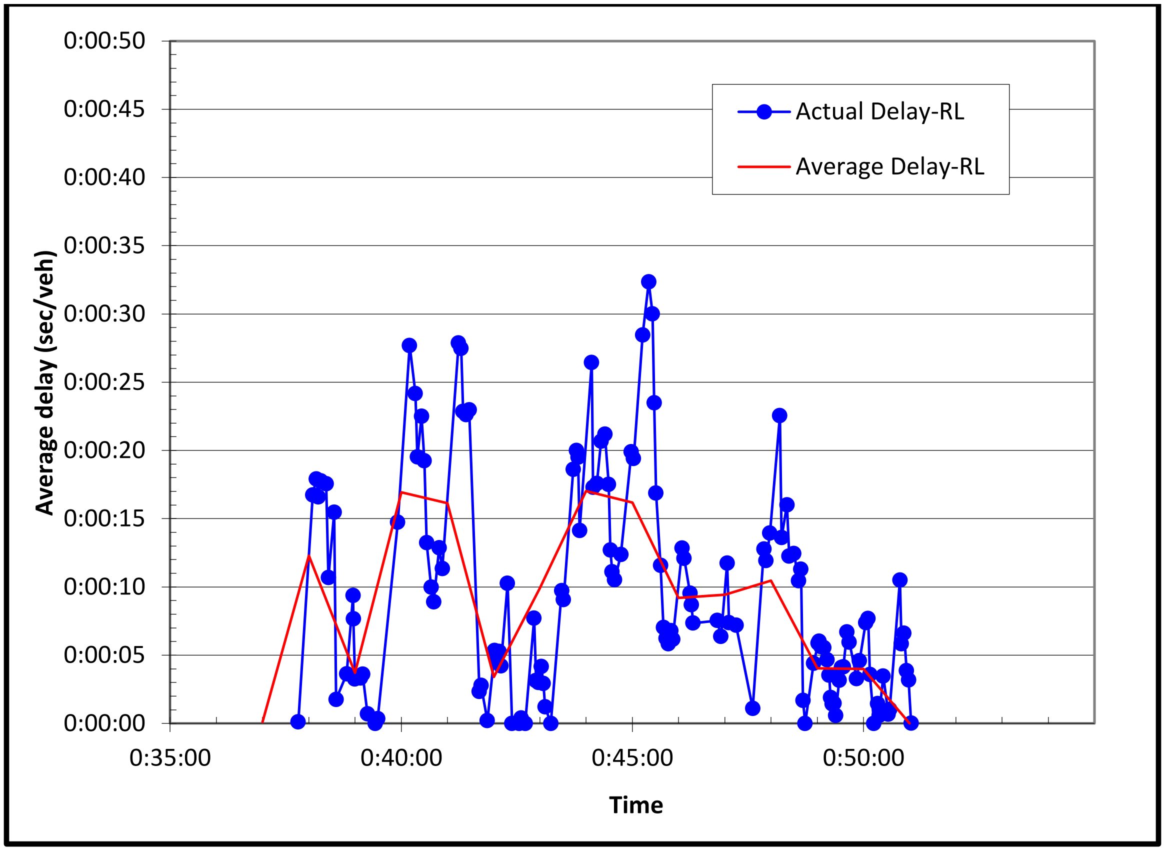

Figure A:4 South Approach Average Delay Chart

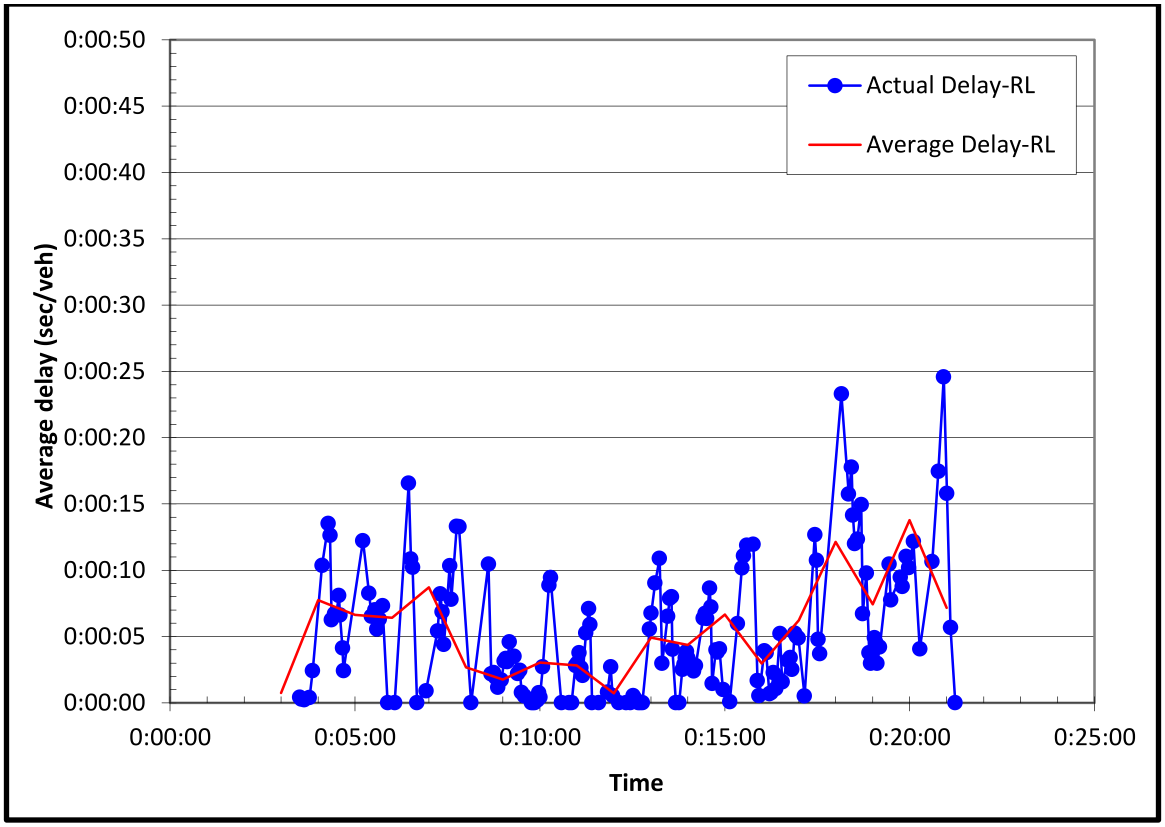

Figure A:5 West Approach Average Delay Chart

Appendix B: Initial VISSIM Output Results for Five Simulation Runs

B-1 Hypothetical Model VISSIM Output Results

Network Performance								
Vehicle Class	**Number of Vehicles**	**Total**			**Avg Speed(mi/h)**	**Per Vehicle**		
		Travel Time(h)	Distance(mi)	Delay(h)		Avg Delay (s)	Avg Number of Stops	Avg Stop Delay (s)
Run 1(1)								
Car (10)	2006	59.05	1725.57	5.62	29.22	10.08	0	0.44
HGV (20)	32	0.94	27.15	0.08	28.95	9.45	0	0.17
Bus (30)	232	6.96	198.21	0.86	28.48	13.36	0	0.72
Total	2270	66.95	1950.94	6.56	29.14	10.41	0	0.46
Run 2(2)								
Car (10)	2026	59	1742.97	5.07	29.54	9.01	0	0.24
HGV (20)	35	1	29.02	0.09	29.04	9.23	0	0.01
Bus (30)	219	6.36	184.49	0.65	29.02	10.67	0	0.25
Total	2280	66.36	1956.48	5.81	29.48	9.17	0	0.24
Run 3(3)								
Car (10)	2033	60.38	1750.35	6.19	28.99	10.95	0	0.41
HGV (20)	32	0.95	27.15	0.11	28.66	12.05	0	0.2
Bus (30)	221	6.59	186.66	0.79	28.31	12.81	0	0.61
Total	2286	67.92	1964.16	7.08	28.92	11.15	0	0.43
Run 4(4)								
Car (10)	2000	58.78	1719.93	5.56	29.26	10.02	0	0.36
HGV (20)	33	1.01	28.69	0.13	28.33	13.68	1	0.63
Bus (30)	215	6.45	184.78	0.73	28.63	12.16	0	0.56
Total	2248	66.25	1933.4	6.42	29.19	10.27	0	0.38
Run 5(5)								
Car (10)	2062	59.81	1772.35	5.06	29.63	8.84	0	0.29
HGV (20)	40	1.17	34.15	0.1	29.23	9.24	0	0.23
Bus (30)	190	5.65	164.22	0.55	29.05	10.44	0	0.3
Total	2292	66.63	1970.71	5.72	29.58	8.98	0	0.29

Table B:1 Hypothetical Model Network Performance

Delay									
Intersection	**Approach**	**Movement**	**Run**					**LOS**	**Average(s)**
			1	**2**	**3**	**4**	**5**		
			Delay(s)	Delay(s)	Delay(s)	Delay(s)	Delay(s)		
entire intersection	NB	Left 2	11.4	6.1	12.5	11.1	6.2		9.5
		Through	14.4	9.4	16.3	14	9.4		12.7
		Right 2	16	10.8	19.6	14.9	13.1		14.8
		Total	14	9	16.1	13.5	9.4		12.4
	EB	Left 2	4.7	4.9	4.5	4.8	4.6		4.7
		Through	7.1	6.2	6.3	6.3	6.2		6.4
		Right 2	8.4	9.9	7.6	8	9		8.6
		Total	6.7	6.9	6.1	6.4	6.4		6.5
	SB	Left 2	4.3	6.1	4.7	4.5	4.7		4.9
		Through	7.1	8.5	7.6	7	7.6		7.6
		Right 2	7.8	10.2	8.6	9.1	8.4		8.8
		Total	6.6	8.3	7.1	6.8	7.2		7.2
	WB	Left 2	4.5	4.6	4.7	4.4	4.6		4.5
		Through	6.4	6.3	7.4	6.6	6.9		6.7
		Right 2	8.8	8.2	9.6	10.2	8.6		9.1
		Total	6.5	6.3	7.3	6.8	6.8		6.7
	Total		9.1	8	10	9.1	7.7	A	8.8
NETWORK TOTAL			9.1	8	10	9.1	7.7		8.8

Table B:2 Hypothetical Model Delay Data

Travel Times											
Movement	TravelTime Section	Distance(ft)	Run					Travel Time			Average Speed (mph)
			1	2	3	4	5	Average(s)	Min(s)	Max(s)	
west entry-south exit	1	92.9	5	5.3	4.5	4.2	4.6	4.7	1.8	9.8	13.5
west entry-east exit	2	203.2	8.7	8.1	8.3	8.3	8.4	8.4	5.3	16	16.5
west entry-north exit	3	314.9	12	13	12	13	12	12	9.2	35	17.5
west entry-west exit	4	421.4	0	0	0	0	0	0	0	0	0
west app-south exit	5	351.1	15	16	14	14	15	15	6.7	43	16.3
west app-east exit	6	461.4	18	17	17	17	17	17	11	50	18.4
west app-north exit	7	572.8	20	20	19	20	20	20	14	44	19.9
west app-west exit	8	679.6	0	0	0	0	0	0	0	0	0
south entry-east exit	9	94	4.1	4.2	4.7	4.6	4.6	4.4	1.8	9.5	14.6
south entry-north exit	10	205.7	8.7	8.2	8.8	8.6	8.4	8.5	5.8	29	16.5
south entry-west exit	11	312.2	11	11	11	11	11	11	8.8	17	19.4
south entry-south exit	12	424.6	0	0	0	0	0	0	0	0	0
south app-east exit	13	345.7	18	17	22	19	18	19	6.7	59	12.7
south app-north exit	14	457.3	22	20	24	23	20	22	11	63	14.4
south app-west exit	15	563.9	23	20	24	23	20	22	14	68	17.7
south app-south exit	16	676.3	0	0	0	0	0	0	0	0	0
east entry-north exit	17	94.6	5.8	5.7	6.1	5.5	5.5	5.7	1.8	27	11.3
east entry-west exit	18	201.2	7.7	7.9	7.7	7.9	7.8	7.8	5.2	13	17.6
east entry-south exit	19	313.6	11	11	11	11	11	11	8.9	17	18.8
east entry-east exit	20	423.9	0	0	0	0	0	0	0	0	0
east app-north exit	21	340.8	15	15	16	16	15	15	6.5	59	15.2
east app-west exit	22	447	16	16	17	17	17	17	10	67	18.4
east app-south exit	23	559.4	19	19	19	18	18	19	14	49	20.6
east app-east exit	24	670.1	0	0	0	0	0	0	0	0	0
north entry-west exit	25	93.9	3.9	4.5	3.8	4.2	4	4.1	1.8	13	15.6
north entry-south exit	26	206.2	7.8	7.9	7.7	7.8	8	7.9	5.5	16	17.8
north entry-east exit	27	316.5	11	12	11	11	11	11	9.2	18	19.3
north entry-north exit	28	428.2	0	0	0	0	0	0	0	0	0
north app-west exit	29	341.1	14	16	14	14	14	14	6.6	39	16.3
north app-south exit	30	453	17	18	17	17	18	17	10	43	17.8
north app-east exit	31	563.8	19	20	19	19	19	19	14	42	20.2
north app-north exit	32	675	0	0	0	0	0	0	0	0	0

Table B:3 Hypothetical Model Travel Time Data

Travel Time Delay									
Movement	**TravelTime Section**	**Run**					**Average(s)**	**Min(s)**	**Max(s)**
		1	**2**	**3**	**4**	**5**			
west app-south exit	5	7.4	8.7	6.7	6.9	7.5	7.4	0	35.4
west app-east exit	6	6.4	5.5	5.7	5.8	5.7	5.9	0.1	39.4
west app-north exit	7	4.5	4.7	4.2	4.5	4.4	4.5	0.2	29.9
north app-south exit	30	6.1	7.3	6.3	6	6.6	6.4	0	31.6
north app-west exit	29	6.4	8.8	6.9	7.2	7	7.2	0	32.7
north app-east exit	31	3.9	5.3	4.1	4	4.2	4.3	0.2	27
south app-west exit	15	8.3	5.7	9.3	8.6	5.5	7.5	0.2	53
south app-east exit	13	11	9.3	15	12	11	11	0	51.7
south app-north exit	14	11	8.5	13	11	8.3	10	0.1	51.7
east app-north exit	21	7.9	7.4	8.8	9.2	7.9	8.2	0	51.4
east app-west exit	22	5.9	5.6	6.8	6.1	6.4	6.2	0.2	57.2
east app-south exit	23	4.1	4.1	4.2	3.9	4.1	4.1	0.4	34.7
west entry-south exit	1	3	3.3	2.5	2.2	2.7	2.7	0	7.8
west entry-north exit	3	2.6	2.7	2.4	2.8	2.5	2.6	0.1	26
west entry-east exit	2	2.9	2.3	2.5	2.4	2.5	2.5	0	10.2
south entry-west exit	11	1.8	1.9	2.3	2.1	2	2	0.2	7.6
south entry-north exit	10	2.7	2.1	2.7	2.5	2.3	2.5	0.1	23.1
south entry-east exit	9	2.1	2.2	2.8	2.6	2.6	2.5	0	7.7
east entry-south exit	19	2.1	2.1	2.1	2.1	2.1	2.1	0.2	7.3
east entry-west exit	18	2.3	2.6	2.4	2.5	2.5	2.5	0.1	7.6
east entry-north exit	17	3.8	3.7	4	3.5	3.5	3.7	0	25.1
north entry-west exit	25	2	2.5	1.8	2.3	2	2.1	0	11
north entry-south exit	26	1.9	2.1	1.9	1.9	2.1	2	0	9.1
north entry-east exit	27	1.5	2	1.5	1.7	1.7	1.7	0.1	7.7

Table B:4 Hypothetical Model Travel Time Delay

Queue Lengths												
	Approach	Movement	95% Queues per Run					Max	95%	Median	Average
			1	2	3	4	5				
			20	22	24	26	28				
entire intersection	NB	U-turn Marker	298	86	296	303	86	842	225.4	0	32.3
		Left 2	298	86	296	303	86	842	225.4	0	32.3
		Through	298	86	296	303	86	842	225.4	0	32.3
		Right 2	298	86	296	303	86	842	225.4	0	32.3
	EB	U-turn Marker	0	0	0	0	0	153	0	0	1.7
		Left 2	0	0	0	0	0	153	0	0	1.7
		Through	0	0	0	0	0	153	0	0	1.7
		Right 2	0	0	0	0	0	153	0	0	1.7
	SB	U-turn Marker	26	46	26	25	26	483	26.9	0	5
		Left 2	26	46	26	25	26	483	26.9	0	5
		Through	26	46	26	25	26	483	26.9	0	5
		Right 2	26	46	26	25	26	483	26.9	0	5
	WB	U-turn Marker	26	0	28	0	26	171	24.3	0	2.7
		Left 2	26	0	28	0	26	171	24.3	0	2.7
		Through	26	0	28	0	26	171	24.3	0	2.7
		Right 2	26	0	28	0	26	171	24.3	0	2.7

Table B:5 Hypothetical Model Queue Lengths

Volumes								
Intersection	**Approach**	**Movement**	**Run**					**Standard Deviation**
			1	**2**	**3**	**4**	**5**	
			20	22	24	26	28	
entire intersection	NB	Left 2	204	194	203	200	198	4
		Through	364	401	366	397	364	18.9
		Right 2	191	206	193	190	180	9.3
		Total	759	801	762	787	742	23.6
	EB	Left 2	99	102	105	97	102	3.1
		Through	149	124	151	140	140	10.7
		Right 2	89	93	86	105	83	8.6
		Total	337	319	342	342	325	10.5
	SB	Left 2	194	203	200	197	171	12.7
		Through	401	366	397	364	435	29.2
		Right 2	206	193	190	180	202	10.3
		Total	801	762	787	741	808	27.9
	WB	Left 2	102	105	97	102	101	2.9
		Through	124	151	140	140	153	11.5
		Right 2	93	86	105	83	113	12.7
		Total	319	342	342	325	367	18.7
	Total		2216	2224	2233	2195	2242	18
NETWORK TOTAL			2216	2224	2233	2195	2242	18

Table B:6 Hypothetical Model Hypothetical Model Flow Data

B-2 Exiting Model VISSIM Output Results

Network Performance								
Vehicle Class	**Number of Vehicles**	**Total**			**Avg Speed(mi/h)**	**Per Vehicle**		
		Travel Time(h)	Distance(mi)	Delay(h)		Avg Delay (s)	Avg Number of Stops	Avg Stop Delay (s)
Run 1(1)								
Car (10)	2537	52.6	1234.91	14.03	23.48	19.91	1	3.19
HGV (20)	36	0.74	17.84	0.17	24.1	17.05	1	3.4
Bus (30)	292	6.54	141.86	2.13	21.68	26.3	1	4.35
Total	2865	59.88	1394.61	16.33	23.29	20.53	1	3.31
Run 2(2)								
Car (10)	2488	51.98	1223.69	13.79	23.54	19.95	1	2.92
HGV (20)	47	0.94	22.91	0.22	24.27	16.91	1	1.56
Bus (30)	267	5.97	131.27	1.85	21.99	25.01	1	3.52
Total	2802	58.89	1377.87	15.86	23.4	20.38	1	2.95
Run 3(3)								
Car (10)	2563	52.74	1258.53	13.43	23.86	18.87	1	3.4
HGV (20)	36	0.8	17.62	0.25	22.02	25.03	1	3.89
Bus (30)	270	5.73	132.42	1.56	23.09	20.82	1	3.63
Total	2869	59.28	1408.57	15.25	23.76	19.13	1	3.43
Run 4(4)								
Car (10)	2491	58.2	1218.88	20.18	20.94	29.16	2	4.62
HGV (20)	49	1.26	23.82	0.52	18.94	37.9	2	5.26
Bus (30)	264	6.78	129.84	2.7	19.14	36.88	2	6.5
Total	2804	66.25	1372.53	23.4	20.72	30.04	2	4.81
Run 5(5)								
Car (10)	2511	46.44	1229.34	8.09	26.47	11.6	1	2.14
HGV (20)	56	1.1	27.65	0.23	25.22	14.92	1	2.83
Bus (30)	254	4.91	123.05	1.04	25.08	14.76	1	2.38
Total	2821	52.44	1380.04	9.36	26.32	11.95	1	2.18

Table B:7 Exiting Model Network Performance

Delay												
Intersection	**Approach**	**Movement**	**Run**					**LOS**	**Average(s)**	**Min(s)**	**Max(s)**
			1	**2**	**3**	**4**	**5**				
			Delay(s)	Delay(s)	Delay(s)	Delay(s)	Delay(s)				
entire intersection	NB	U-turn Marker	41	29	26	59	11		32.5	0.9	119.5
		Left 2	38	37	34	60	15		36.6	0.3	135.1
		Through	41	40	39	62	18		40	0.2	140.7
		Right 2	43	39	38	65	18		40.8	0.2	144.9
		Total	40	39	37	62	17		39	0.2	144.9
	EB	U-turn Marker	6.3	8.1	8.8	6.2	6.8		7.2	0.2	34.8
		Left 2	9	7.6	9.3	7.6	10		8.8	0.2	58.8
		Through	11	9.7	12	9.8	12		10.7	0.1	68.9
		Right 2	13	11	13	12	14		12.8	0	68.5
		Total	11	9.8	12	10	12		11	0	68.9
	SB	U-turn Marker	1.1	1	1.1	1.3	1.1		1.1	0.2	6.1
		Left 2	1.4	1.4	1.2	1.2	1.1		1.3	0.3	10.6
		Through	2	1.7	1.7	1.7	1.7		1.8	0.1	18.3
		Right 2	2.7	3.1	2.5	2.7	3.1		2.8	0	31.9
		Total	2.1	2	1.8	1.8	1.9		1.9	0	31.9
	WB	Left 2	6.9	8.5	7	7.2	7.2		7.4	0.5	44.5
		Through	7.8	11	9.5	8.1	8.9		9.1	0.1	56.5
		Right 2	6.5	14	11	9.6	10		10.3	0	47.8
		Total	7.2	11	8.8	8	8.5		8.6	0	56.5
	Total		19	20	19	27	11	B	19.2	0	144.9
NETWORK TOTAL			19	20	19	27	11		19.2	0	144.9

Table B:8 Exiting Model Delay Data

Travel Times												
Movement	Section	Distance(ft)	Run					Travel Time			Average Speed (mph)
			1	2	3	4	5	Average(s)	Min(s)	Max(s)	
west entry-south exit	1	96.7	4.5	4.5	4.6	4.3	4.6	4.5	1.8	28	14.7
west entry-east exit	2	206	8.5	8.4	8.3	8.6	8.7	8.5	5.4	32.3	16.5
west entry-north exit	3	317	13.3	12.9	13.3	12.9	13.4	13.2	9.2	48	16.4
west entry-west exit	4	426	14.6	15.4	16.1	15.5	15.3	15.4	12.2	36.5	18.8
west app-south exit	5	343	18.3	17	19.1	17.3	19.3	18.3	6.5	70.2	12.8
west app-east exit	6	450	20.8	20	21.8	20	22.3	21	10.3	78.4	14.6
west app-north exit	7	562	23.5	22.1	23.8	22	24.9	23.3	14	72.8	16.4
west app-west exit	8	669	23.4	25.2	25.5	23.8	23.9	24.3	17.4	53.2	18.8
south entry-east exit	9	96	5.2	5.1	5.7	5.8	5.2	5.4	1.9	37.6	12.1
south entry-north exit	10	208	11.5	10.8	11.5	11.6	10.3	11.2	5.9	44.8	12.7
south entry-west exit	11	317	14	13.3	13.4	14.1	12.8	13.5	8.8	38.5	16
south entry-south exit	12	430	19.9	18.5	17.4	17.9	16.5	18.1	13	36	16.2
south app-east exit	13	339	39.2	34.6	38.2	49.4	23.2	37.2	7.4	109	6.2
south app-north exit	14	449	42.8	39.5	44.4	52.4	27.9	41.6	11.1	109	7.4
south app-west exit	15	557	44.1	39.9	42.8	54.1	28.8	41.8	13.8	109	9.1
south app-south exit	16	671	49.5	40.9	39.2	54.2	28.1	42.1	18.6	96.8	10.9
east entry-north exit	17	96.7	4.7	7.4	6.2	6.7	6.6	6.3	1.8	37.5	10.5
east entry-west exit	18	206	8.7	9.6	9.7	9	8.8	9.2	5.2	39.1	15.2
east entry-south exit	19	319	12.8	13.6	12.8	12.6	12.8	12.9	9.2	50.8	16.9
east entry-east exit	20	426	0	0	0	0	0	0	0	0	0
east app-north exit	21	343	12.6	20.4	16.7	15.8	16.2	16.4	7	53.9	14.2
east app-west exit	22	450	17.6	20.7	19.5	17.9	18.7	18.9	10.3	64.8	16.2
east app-south exit	23	562	20.6	22.3	21	21	20.9	21.2	14	59	18.1
east app-east exit	24	671	0	0	0	0	0	0	0	0	0
north entry-west exit	25	100	2.7	2.7	2.6	2.8	2.6	2.7	1.9	10.5	25.3
north entry-south exit	26	213	6.6	6.6	6.6	6.6	6.6	6.6	5.7	15.5	22
north entry-east exit	27	322	10.2	10.2	10.2	10	10	10.1	9.3	19.5	21.7
north entry-north exit	28	434	14.2	14	14.1	14.2	13.8	14	12.8	19	21.1
north app-west exit	29	352	8.7	8.7	8.5	8.7	8.6	8.7	7	18.6	27.6
north app-south exit	30	464	12.5	12.5	12.4	12.4	12.4	12.5	10.8	21.8	25.3
north app-east exit	31	573	16	16	16	15.7	15.9	15.9	14.4	26.4	24.6
north app-north exit	32	685	20	19.9	19.9	19.9	19.5	19.8	17.6	24.7	23.6

Table B:9 Exiting Model Travel Times

Travel Time Delay									
Name	**TravelTime Section**	**Run**					**Travel Time Delay (s)**		
		1	**2**	**3**	**4**	**5**	**Average(s)**	**Min(s)**	**Max(s)**
west app-south exit	5	11	9.9	12	10	12	11	0	63.6
west app-east exit	6	9.8	9	11	9.1	11	10	0.1	67.5
west app-north exit	7	8.6	7.2	8.9	7.1	10	8.4	0.2	58.6
west app-west exit	8	5.7	7.5	7.7	5.9	6.3	6.6	0.2	34.2
south app-south exit	16	32	23	21	36	9.9	24	0.7	77.7
south app-east exit	13	32	28	31	42	16	30	0.2	103
south app-west exit	15	30	26	29	40	15	28	0.3	94.4
south app-north exit	14	32	28	33	41	17	31	0.1	98
east app-south exit	23	6.1	7.9	6.5	6.5	6.5	6.7	0.4	44.1
east app-west exit	22	7	10	8.9	7.4	8.1	8.3	0.1	54.6
east app-north exit	21	5.5	13	9.6	8.7	9.1	9.3	0	46.5
north app-north exit	32	1	0.8	0.9	1.2	1	1	0.2	6.1
north app-east exit	31	0.9	1	0.9	0.8	0.8	0.9	0.2	10.5
north app-south exit	30	1.1	1.1	1	1	1	1.1	0	9.9
north app-west exit	29	1.4	1.4	1.1	1.4	1.2	1.3	0	11
west entry-east exit	2	2.6	2.5	2.4	2.7	2.8	2.6	0	25.9
west entry-north exit	3	3.5	3.1	3.5	3.1	3.6	3.4	0.1	37.6
west entry-west exit	4	1.9	2.7	3.4	2.6	2.7	2.7	0.1	24.4
west entry-south exit	1	2.4	2.4	2.6	2.3	2.6	2.5	0	25.8
south entry-south exit	12	7	5.3	4.5	5	3.3	5	0.6	22.6
south entry-east exit	9	3.2	3.1	3.7	3.8	3.2	3.4	0	35.7
south entry-north exit	10	5.3	4.7	5.4	5.5	4.2	5	0.1	38.5
south entry-west exit	11	4.9	4.1	4.3	5	3.6	4.4	0.2	29.6
east entry-north exit	17	2.7	5.3	4.1	4.6	4.5	4.3	0	35.6
east entry-west exit	18	3.2	4.1	4.1	3.5	3.3	3.6	0	33.2
east entry-south exit	19	3.3	4.2	3.3	3.1	3.4	3.5	0.2	41
north entry-north exit	28	0.6	0.4	0.5	0.8	0.5	0.5	0.2	5.4
north entry-west exit	25	0.6	0.6	0.5	0.7	0.5	0.6	0	8.2
north entry-south exit	26	0.5	0.5	0.5	0.4	0.5	0.5	0	9.1
north entry-east exit	27	0.4	0.5	0.4	0.4	0.3	0.4	0.1	9.2

Table B:10 Exiting Model Travel Time Delay

Queue Lengths												
Intersection	**Approach**	**Movement**	**95% Queues per Run**									
			1	**2**	**3**	**4**	**5**	**Max**	**95%**	**Median**	**Average**
			20	22	24	26	28				
entire intersection	NB	U-turn Marker	777	760	427	1155	159	1327	807	100.8	191.8
		Left 2	777	760	427	1155	159	1327	807	100.8	191.8
		Through	777	760	427	1155	159	1327	807	100.8	191.8
		Right 2	777	760	427	1155	159	1327	807	100.8	191.8
	EB	U-turn Marker	90	76	99	76.6	97	241.5	89.9	0	17.7
		Left 2	90	76	99	76.6	97	241.5	89.9	0	17.7
		Through	90	76	99	76.6	97	241.5	89.9	0	17.7
		Right 2	90	76	99	76.6	97	241.5	89.9	0	17.7
	SB	U-turn Marker	0	0	0	0	0	32.9	0	0	0
		Left 2	0	0	0	0	0	32.9	0	0	0
		Through	0	0	0	0	0	32.9	0	0	0
		Right 2	0	0	0	0	0	32.9	0	0	0
	WB	U-turn Marker	19	52	39	39.5	36	121.9	39.2	0	5.5
		Left 2	19	52	39	39.5	36	121.9	39.2	0	5.5
		Through	19	52	39	39.5	36	121.9	39.2	0	5.5
		Right 2	19	52	39	39.5	36	121.9	39.2	0	5.5

Table B:11 Exiting Model Queue Lengths

Volumes								
Intersection	**Approach**	**Movement**	**Run**					**Standard Deviation**
			1	**2**	**3**	**4**	**5**	
			20	22	24	26	28	
entire intersection	NB	U-turn Marker	25	15	22	17	22	4.1
		Left 2	330	336	332	320	338	7
		Through	351	391	381	374	353	17.5
		Right 2	292	313	298	300	264	18.1
		Total	998	1055	1033	1011	977	30.3
	EB	U-turn Marker	31	39	30	34	32	3.6
		Left 2	194	200	213	201	206	7.1
		Through	267	217	248	254	229	20
		Right 2	350	314	336	318	338	14.9
		Total	842	770	827	807	805	27.1
	SB	U-turn Marker	14	26	13	14	16	5.4
		Left 2	69	59	71	74	61	6.5
		Through	327	294	327	296	316	16.2
		Right 2	126	120	102	104	116	10.3
		Total	536	499	513	488	509	17.9
	WB	Left 2	164	169	158	180	167	8.1
		Through	180	193	205	188	213	13.2
		Right 2	67	72	81	69	84	7.5
		Total	411	434	444	437	464	19.1
	Total		2787	2758	2817	2743	2755	29.9
NETWORK TOTAL			2787	2758	2817	2743	2755	29.9

Table B:12 Exiting Model Flow Data

Appendix C: Existing Model Calibration

Note:

- Gap time and Headway were adjusted using field data used in the NCHRP 572
- The field data used in the NCHRP 572 was missing delay records for the southbound traffic so the travel time data was used to calibrate existing VISSIM model. Table C:1 shows the VISSIM Travel Time Data for the different trials.
- Driver behavior was adjusted using ranges from Table 4:17. For each trial different sets of driver behavior parameters were adjusted until an there was not significant change in subsequent trial. After all parameters had been adjusted, the gap time and headway were also adjusted in a similar fashion. Table C:2 shows the parameters adjusted for each trial. Figure C:1 shows the average error that was observed per trial.

Movement	Field data- Travel Time (s)			VISSIM Data								
				Travel Time Average(s)								
	Right Lane	Left Lane	Average	Trial 1	Trial 2	Trial 3	Trial 4	Trial 5	Trial 6	Trial 7	Trial 8	Trial 9
W-S	3.6	5.8	4.7	4.3	4.2	4.5	4.3	4.3	4.4	4.4	4.5	4.5
W-E	8.6	8.8	8.7	8.4	8.4	8.4	8.3	8.3	8.5	8.4	8.4	8.4
W-N	14.7	13.3	14	13	13.1	13	13.3	13.3	13.2	13.2	13.3	13.3
S-N	7.9	7.6	7.75	9.5	11.2	11.1	11.3	11.3	11	11.2	11.3	11.3
S-W	13.3	13.5	13.4	11.9	13.4	13.5	13.4	13.4	13.3	13.5	13.3	13.3
S-S	16.7	17.8	17.25	15.8	17.9	17.9	17.3	17.3	17.6	17.3	17.4	17.4
E-W	8.6	10	9.3	10	9	9.2	9	9	9.1	9.2	9.4	9.4
E-S	13.7	13.8	13.75	13.8	12.8	12.9	12.9	12.9	12.9	12.9	12.9	12.9
N-S	8.9	9.6	9.25	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.6	6.6
N-E	12.9	12.9	12.9	10.2	10.1	10.2	10.1	10.1	10.2	10.2	10.2	10.2

Table C: 1 VISSIM Travel Time Data

Trial	Simulation period	Random seed	Random seed increment	# of runs	Reduced Speed Area	Gap time	Headway	Driver Behavior (Table 4:17)
1	3600	20	2	17	circulatory path	Default	Default	Default
2	3600	20	2	17	circulatory path	Default	Default	- Average Standstill Distance, - Additive Part of Desired Safety Distance, - Multiplicative Part of Desired Safety Distance
3	3600	20	2	17	circulatory path	Default	Default	- Safety Distance Reduction Factor, - Emergency Stop Distance, - Lane Change Distance
4	3600	20	2	17	circulatory path	Default	Default	- Max Deceleration (Trailing), - Accepted Deceleration (Trailing), - Max. Deceleration for Cooperative Braking
5	3600	20	2	17	circulatory path	Default	Default	- Max Deceleration (Own), - Accepted - Deceleration (Own), -1 ft/s2 per Distance (Own),
6	3600	20	2	17	circulatory path	Field Data	Default	
7	3600	20	2	17	circulatory path	Default	Field Data	
8	3600	20	2	17	circulatory path	Field Data	Field Data	
9	3600	20	2	17	circulatory path	Field Data	Field Data	

Table C:2 Calibration Parameters per Trial

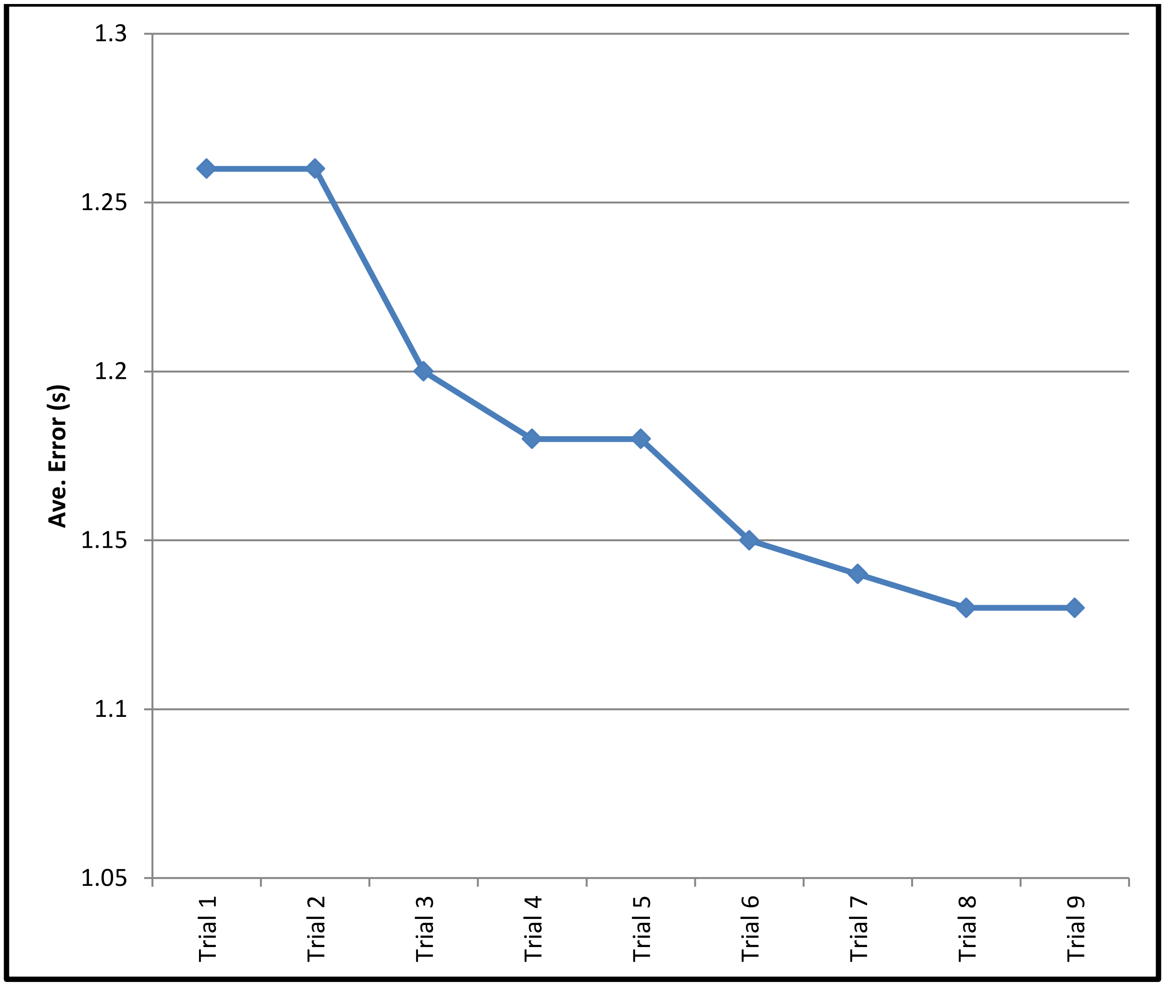

Figure C:1 Plot of Average Error per Trial

Appendix D: T Distribution Table and Passenger Car Equivalent Table

one-tail	**0.5**	**0.25**	**0.2**	**0.15**	**0.1**	**0.05**	**0.025**	**0.01**	**0.005**	**0.001**	**0.0005**
two-tails	**1**	**0.5**	**0.4**	**0.3**	**0.2**	**0.1**	**0.05**	**0.02**	**0.01**	**0.002**	**0.001**
df											
1	0	1	1.376	1.963	3.078	6.314	12.71	31.82	63.66	318.31	636.62
2	0	0.816	1.061	1.386	1.886	2.92	4.303	6.965	9.925	22.327	31.599
3	0	0.765	0.978	1.25	1.638	2.353	3.182	4.541	5.841	10.215	12.924
4	0	0.741	0.941	1.19	1.533	2.132	2.776	3.747	4.604	7.173	8.61
5	0	0.727	0.92	1.156	1.476	2.015	2.571	3.365	4.032	5.893	6.869
6	0	0.718	0.906	1.134	1.44	1.943	2.447	3.143	3.707	5.208	5.959
7	0	0.711	0.896	1.119	1.415	1.895	2.365	2.998	3.499	4.785	5.408
8	0	0.706	0.889	1.108	1.397	1.86	2.306	2.896	3.355	4.501	5.041
9	0	0.703	0.883	1.1	1.383	1.833	2.262	2.821	3.25	4.297	4.781
10	0	0.7	0.879	1.093	1.372	1.812	2.228	2.764	3.169	4.144	4.587
11	0	0.697	0.876	1.088	1.363	1.796	2.201	2.718	3.106	4.025	4.437
12	0	0.695	0.873	1.083	1.356	1.782	2.179	2.681	3.055	3.93	4.318
13	0	0.694	0.87	1.079	1.35	1.771	2.16	2.65	3.012	3.852	4.221
14	0	0.692	0.868	1.076	1.345	1.761	2.145	2.624	2.977	3.787	4.14
15	0	0.691	0.866	1.074	1.341	1.753	2.131	2.602	2.947	3.733	4.073
16	0	0.69	0.865	1.071	1.337	1.746	2.12	2.583	2.921	3.686	4.015
17	0	0.689	0.863	1.069	1.333	1.74	2.11	2.567	2.898	3.646	3.965
18	0	0.688	0.862	1.067	1.33	1.734	2.101	2.552	2.878	3.61	3.922
19	0	0.688	0.861	1.066	1.328	1.729	2.093	2.539	2.861	3.579	3.883
20	0	0.687	0.86	1.064	1.325	1.725	2.086	2.528	2.845	3.552	3.85
21	0	0.686	0.859	1.063	1.323	1.721	2.08	2.518	2.831	3.527	3.819
22	0	0.686	0.858	1.061	1.321	1.717	2.074	2.508	2.819	3.505	3.792
23	0	0.685	0.858	1.06	1.319	1.714	2.069	2.5	2.807	3.485	3.768
24	0	0.685	0.857	1.059	1.318	1.711	2.064	2.492	2.797	3.467	3.745
25	0	0.684	0.856	1.058	1.316	1.708	2.06	2.485	2.787	3.45	3.725
26	0	0.684	0.856	1.058	1.315	1.706	2.056	2.479	2.779	3.435	3.707
27	0	0.684	0.855	1.057	1.314	1.703	2.052	2.473	2.771	3.421	3.69
28	0	0.683	0.855	1.056	1.313	1.701	2.048	2.467	2.763	3.408	3.674
29	0	0.683	0.854	1.055	1.311	1.699	2.045	2.462	2.756	3.396	3.659
30	0	0.683	0.854	1.055	1.31	1.697	2.042	2.457	2.75	3.385	3.646
40	0	0.681	0.851	1.05	1.303	1.684	2.021	2.423	2.704	3.307	3.551
60	0	0.679	0.848	1.045	1.296	1.671	2	2.39	2.66	3.232	3.46
80	0	0.678	0.846	1.043	1.292	1.664	1.99	2.374	2.639	3.195	3.416
100	0	0.677	0.845	1.042	1.29	1.66	1.984	2.364	2.626	3.174	3.39
1000	0	0.675	0.842	1.037	1.282	1.646	1.962	2.33	2.581	3.098	3.3
z	0	0.674	0.842	1.036	1.282	1.645	1.96	2.326	2.576	3.09	3.291
	0%	50%	60%	70%	80%	90%	95%	98%	99%	99.80%	99.90%
	Confidence Level										

Table D: 1 T Distribution Table

Vehicle Type	Passenger Car Equivalent, E_T
Passenger Car	1
Heavy Vehicle	2
Bicycle	0.5

Table D: 2 Passenger Car Equivalent Table

BIBLIOGRAPHY

AASHTO (2004). “A Policy on Geometric Design of Highways and Streets.” American Association of State Highway and Transportation Officials, 5th edition, Washington, D.C.

Brackstone, M. and M. McDonald, *Car-following: a historical review.* Transportation Research, 1999. 2F: p. 181-196.

Barcelo.J. et al (2004), “ Microscopic traffic simulation: A tool for the design, analysis andevaluation of intelligent transport systems” Journal of Intelligent and Robotic Systems, Volume 41, Numbers 2-3, January 2005 , pp. 173-203(31)

Dion, F., H. Rakha, and Y.-S. Kang, *Comparison of delay estimates at under-saturated and over-saturated pre-timed signalized intersections.* Transportation Research Part B Methodological, 2004. 38(NCHRP 2010) : p. 99-122.

FHWA (2004b). US Department of Transportation, Federal Highway Administration, “Traffic Analysis Toolbox Volume III: Guidelines for Applying Traffic Microsimulation Software.”, Publication No. *FHWA-HRT-04-040, 2004*. Washington, D.C. Web site, http://ops.fhwa.dot.gov/trafficanalysistools/tat_vol3/foreword.htm
Accessed December 1, 2010.

G.Duncan (2000), Paramics Technical Report:Car-Following, Lane-Changing and Junction Modelling.

Gipps, P.G., *A behavioral car-following model for computer simulation.* Transportation Research, 1981. 15B: p. 105-111.

Hagring, O. (2001). “Derivation of capacity equation for roundabout entry with mixed circulating and exiting flows.” *Transportation Research Record: Journal of the Transportation Research Board, No. 1776,* Transportation Research Board of the National Academies, Washington, D.C., pp. 91-99.

Johnson, G. P. (2006). “Roundabout Analysis at Idaho Maryland/E. Main Intersection Using VISSIM Microsimulation Model Final Report”. For the Nevada Country Transportation Commission. PRISM Engineering Consulting, Fresno, California.

Loren Bloomberg and Jim Dale (2000) “A Comparison of the VISSIM and CORSIM Traffic Simulation Models on a Congested Network”

Rakha H. and Crowther B. (2003), “Comparison and Calibration of FRESIM and INTEGRATION Steady-state Car-following Behavior”, *Transportation Research* , 37A, pp. 127

Russell, E., Rys, M., and Luttrell, G. (2000). “Modeling Traffic Flows and Conflicts at Roundabouts.” Funded by: Mack-Blackwell Rural Transportation Center, University of Arkansas, Kansas State University, City of Manhattan, Kansas.

Roess, R., Prassas, E., McShane, W. *Traffic Engineering* - 4th Edition, Prentice Hall, July 4th, 2010.

Sisiopiku V., and Heung-Un, O. (2001). “Evaluation of Roundabout Perfromance Using SIDRA.” Journal of Transportation Engineering. March/April issue, pp 143-150.

SIDRA User’s Manual (2009). Akcelik and Associates Pty Ltd, PO Box 1075G, Greythorn, Vic 3104, Australia.

Montgomery, C., Runger, G., Hubele N. *Engineering Statistics* – 3rd Edition, Wiley, July 4, 2003.

TEH (1992). *Traffic Engineering Handbook*, Fourth Edition, Editor by Pline, J. L. Institute of Transportation Engineers, Prentice Hall, NJ. 1992.

WISDOT (2008). Wisconsin Department of Transportation. Roundabout Guide, web site, http://www.dot.wisconsin.gov/safety/motorist/roaddesign/roundabouts/docs/guide-wisdotrab.pdf. Accessed February 3, 2013