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OPERATIONAL PERFORMANCE OF KANSAS ROUNDABOUTS: PHASE II

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16 Abstract

Modern roundabouts are being implemented throughout the United States (US) in a variety of locations. Single-lane roundabouts may perform better than two-way stop-controlled (TWSC) intersections in the US under some conditions. The safety record of well designed modern roundabouts is excellent.

The primary objective of this study was to compare the operational performance of 11 modern roundabouts in Kansas with other intersection traffic control devices (TCDs) in five locations in Kansas. This study found that there were statistically significant reductions in delay; queuing and proportion of vehicles stopped at all the study sites after the installation of a modern roundabout.

It is reasonable to suggest that the movement of traffic through these intersections should be significantly improved and a modern roundabout should be the best intersection alternative for several other locations in Kansas with similar traffic volumes and similar geometrics. Further studies should be conducted in other locations in Kansas with different traffic conditions and geometrics, particularly those where volumes are high enough that a multi-lane roundabout is required, in order to get a clearer picture.

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OPERATIONAL PERFORMANCE OF KANSAS ROUNDABOUTS: PHASE II

Final Report

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PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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ABSTRACT

Modern roundabouts are being implemented throughout the United States (US) in a variety of locations. Many states and cities are considering roundabouts as a viable alternative to other Traffic Control Devices (TCD's), and, in some cases, complex freeway interchanges. The modern roundabout was developed in the United Kingdom (UK) to eliminate the problems associated with old traffic circles. These modern roundabouts have been in widespread use in other countries since the late 1960's and have been very successful.

The people of US were introduced to traffic circles in 1905. As traffic volumes increased these traffic circles had high crash and/or congestion experiences, and they fell out of favor around the 1950's. The first modern roundabout built in the US, was built in 1990. Since then their application in the US has received increased attention by both the public and transportation professionals. A lack of sufficient information on roundabout operation and design under local conditions and confusion of the general public with early traffic circles have been factors affecting the growth rate of roundabouts in the US.

Single-lane roundabouts may perform better than two-way stop-controlled (TWSC) intersections in the US under some conditions. The safety record of well designed modern roundabouts is excellent. A major US study conducted by the Insurance Institute for Highway Safety (IIHS) evaluated the changes in motor vehicle crashes following conversion of 23 intersections from stop sign and traffic signal control to modern roundabouts. This study estimated reductions of approximately 40% for all crash severities combined, 80% for all injury crashes and 90% for fatal and incapacitating injury crashes.

Safety appears to be better at small and medium capacity roundabouts than at large, multilane roundabouts. Crash reductions at modern roundabouts are most pronounced for motor vehicles, less pronounced for pedestrians, and indefinite for bicyclists, depending on the study and bicycle design treatments.

The primary objective of this study was to compare the operational performance of 11 modern roundabouts in Kansas with other intersection traffic control devices (TCDs) in five locations in Kansas; namely, Olathe, Lawrence, Paola, Newton (2), Topeka (3). Although not a part of the Phase II study, summaries of previous studies of roundabouts in Hutchinson and Manhattan were included in this report for completeness. The operation of the roadways at these intersections was videotaped and traffic flow data was extracted from the videotapes and analyzed using SIDRA (Signalized and Un-signalized Intersection Design and Research Aid) software, version 1.0. The software produces many Measures of Effectiveness (MOEs) of which six were chosen for analyzing the operational performance of roundabouts; namely, average intersection delay, maximum approach delay, 95% queue length, degree of saturation, proportion of vehicles stopped at intersection and maximum proportion of vehicles stopped on an approach.

Results of earlier studies of the first modern roundabout in Kansas (Candlewood and Gary Streets in Manhattan, Kansas) showed that a single-lane, modern roundabout operated better than two-way or four-way stop controls. A modern roundabout at Severance and 23rd streets in Hutchinson, Kansas, showed that the roundabout operated more efficiently than the two-way stop it replaced and more efficiently than a four-way stop and signal control would have. The results from all the sites have been averaged to give an overall picture of the operational performance of roundabouts in Kansas.

This study found that there were statistically significant reductions in delay; queuing and proportion of vehicles stopped at all the study sites after the installation of a modern roundabout. Tables showing the reductions at each of the sites studied are contained in the report. The overall average of results on the six variables used in the study are shown in Table A-1, which shows the averaged results from all sites studied (For 11 Kansas roundabouts including Manhattan and Hutchinson). It is reasonable to suggest that the movement of traffic through these intersections should be significantly improved and a modern roundabout should be the best intersection alternative for several other locations in Kansas with similar traffic volumes and similar geometrics. Further studies should be conducted in other locations in Kansas with different traffic conditions and geometrics, particularly those where volumes are high enough that a multi-lane roundabout is required, in order to get a clearer picture.

TABLE A-1: Kansas Average Results Table [1]

Measures of Effectiveness	Before [2]	R.A [3]	% Diff.	Stat. Diff [4]
Average Intersection Delay (Sec/veh)	20.2	8.0	-65%	Yes
Maximum Approach Delay (Sec/veh)	34.4	10.4	-71%	Yes
95% Queue Length (Feet)	190	104	-44%	Yes
Degree of Saturation (V/C) Intersection	0.463	0.223	-53%	Yes
Proportion of vehicles Stopped (%) Intersection	58	29	-52%	Yes
Max. Proportion of vehicles Stopped (%) Approach	62	37	-42%	Yes

1: 11 Roundabouts with AM and PM combined,

Olathe: Ridgeview/Sheridan, Rogers/Sheridan (Before condition: AWSC) [2 sites]
Topeka: Rice Road North and South (Before condition: Theoretical TWSC) [2 sites]
: US-75/NW 46th Street (Before condition: Traffic Signal) [1 site]
Newton: I-135/Broadway, I-135/First Street (Before condition: Theoretical Traffic Signal) [2 sites]
Lawrence: Harvard Road/Monterey Way (Before condition: AWSC) [1 site]
Paola: Old K.C road/K-68 (Before condition: AWSC) [1 site]
Manhattan: Gary/Candlewood (Before condition: TWSC) [1 site]
Hutchinson: 23rd street/Severance Avenue (Before condition: TWSC) [1 site]

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2. Before: AWSC/TWSC/Signal [AWSC: All-Way Stop control, TWSC: Two-Way Stop control]
3. R.A: Roundabout,
4. Stat. Diff: Statistically Different

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Chapter 1

Introduction and Objectives

1.1 Introduction

The people of United States (US) were introduced to traffic circles, sometimes called rotaries or gyratories, in 1905, and since then many large circles or rotaries were built in the US, mostly in the eastern US. These kinds of circles or rotaries lasted until they fell out of favor around the 1950's due to high crash and/or congestion experiences as traffic volumes increased.

A modern roundabout gives priority to vehicles on the circulating roadway and requires entering vehicles to yield until a suitable gap in the circulating traffic is available. They are generally smaller than the old circles and are designed for low speed operation, achieved by proper deflection. Deflection for entering traffic starts with a splitter island, usually raised, and continues with traffic being directed or deflected around a raised central island. The entering roadway may be widened or flared to assist entry or increase capacity. However, the key to modern roundabout safety is low speed due to deflection, which is a function of the design and placement of key geometric elements as discussed in this report.

Figure 1.1 shows a picture of one of the modern roundabouts in Kansas included in this study. Figure 1.2 shows a schematic diagram that illustrates various parts of a modern roundabout. The modern roundabout was developed in the United Kingdom (UK) to eliminate the problems associated with old traffic circles and early roundabout designs. The UK was the first to develop standards for what they called a "normal roundabout", or the "modern roundabout", and slowly many other countries in the world started recognizing the benefits of this form of intersection traffic control. This progression started in 1966 when they introduced the "off side priority rule" (yield at entry) and continued in a series of changes to 1983 when they published UK standards for a normal roundabout. [Brown 1995] The first modern roundabout built in the US, was built in 1990. Thus, any circular intersection built in the US prior to 1990 is most likely not a modern roundabout (unless by chance) and its operation should not be compared to that of a modern roundabout. Likewise, circles or rotaries built worldwide prior to the mid-1980's are most likely not built to modern roundabout standards.

Modern roundabouts have been a great success in the UK, Europe and Australia and at many intersections are a better alternative than conventional intersection traffic control types such as stop control, yield control and traffic signal control [Austroads 1993; Brown 1995]. Many studies have found that one of the benefits of modern roundabout installation is an improvement in overall safety performance when compared with any other form of intersection traffic control. A major US study [IIHS, 2000] concluded that modern roundabouts decrease all crashes about 39%, injury crashes about 76%, and the study projected a 90% decrease in fatal crashes. [Persaud, et.al.,2001]. Multi-lane roundabouts may have less crash reduction than single lane roundabouts but evidence is mounting that multi-lane modern roundabouts also have superior safety records. These modern roundabouts may not be a common type of intersection in the US, but they're becoming more common, and therefore more familiar, as evidence of their benefits grows.

1.2 Objectives of this Study

The primary objective of this study was to compare the operational performance of several modern roundabouts in Kansas with other intersection traffic control devices (TCDs). This report focuses on seven cities in Kansas: Olathe, Lawrence, Paola, Newton, Topeka, Hutchinson and Manhattan. Results of earlier studies, of the first modern roundabout in Kansas at Candlewood and Gary in Manhattan, and a modern roundabout at Severance and 23rd streets in Hutchinson, have also been included in this report. Detailed reports on earlier studies are available from the Mack Blackwell (National) Transportation Center (MBTC) website http://www.mackblackwell.org/ or from Kansas State University.

FIGURE 1.1: Picture of Modern Roundabout in Hutchinson, Kansas

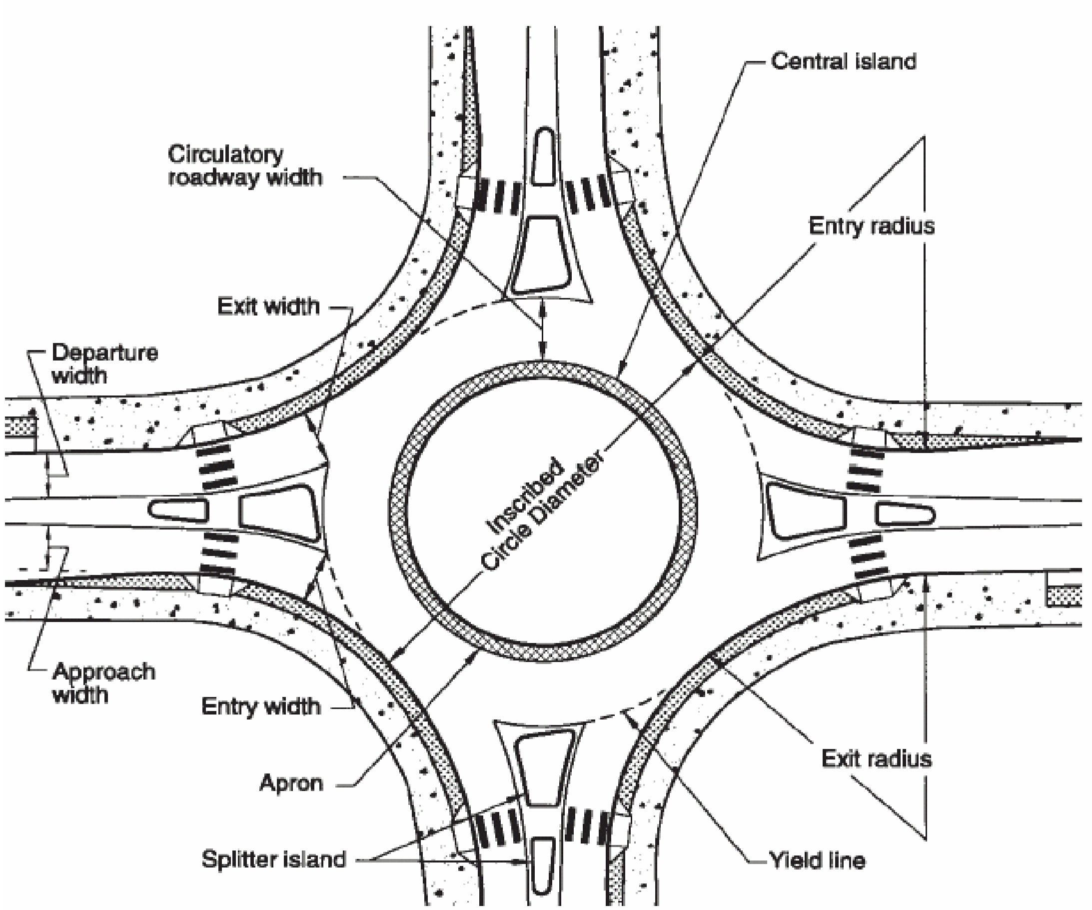


FIGURE 1.2: Geometric Elements of a Modern Roundabout

(Source: FHWA Roundabout guide)

Chapter 2

Background and Review of Literature

2.1 General

This chapter presents a brief history of roundabouts and summarizes key literature reviewed relative to this study.

2.2 Brief History of Roundabouts

Traffic circles were introduced to the people of United States around 1905, when William Phelps Eno designed Columbus Circle, in New York City. As traffic increased, these traffic circles started having congestion problems with circulating traffic which many times led to “locking”. As a result, the public was not happy with them.

In 1929, Eno recognized that the problem of congestion in traffic circles was due to the high volume of traffic and pointed out that the main drawback could be due to the yield-to-right rule, which meant that vehicles in the traffic circle yielded to the entering traffic. He recommended a yield-to-left rule, which would require entering vehicles to yield to the circulating traffic. However, his recommendations were ignored and, in an attempt to solve the locking problem, design philosophy was to design larger rotaries with long weaving sections and longer storage distances between successive entries. Geometry that allowed high-speed entry was common. The right-of-way rule, giving priority to entering vehicles, remained the law. The locking problem became worse as traffic volumes continued to increase. These larger circles had other negative effects such as high-speed entering vehicles, higher speeds on the circulating roadways and, high-speed weaving maneuvers. These characteristics increased crash risks and crashes. Finally, reluctant to reverse the right-of-way rule, and unable to solve operational and locking problems, traffic circles fell out of favor in the US around the mid 1950's. In the UK, in the 1960's traffic engineers were ready to give up on them also.

FIGURE 2.1: View of Columbus Circle, Circa 1915
(Courtesy: New York Department of Planning, in Jacquemart 1998)

UK researchers conducted a lot of research to overcome the congestion and locking problems in circles with high volumes. In 1966 they adopted a mandatory "give-way" rule at all circular intersection, which required entering traffic to give-way or yield to the circulating traffic. This rule was known as the "offside priority" or "yield-at-entry" rule.

This rule almost immediately ended the locking problems and ended most of the then existing problems with the old circles. Further research in the UK proved that the offside priority rule would eliminate locking problems, increase capacity, reduce delay and also increase safety [Todd 1988, cited in Jacquemart 1998]. The UK further developed design guidelines in the 1970's and 80's for design and deflection, resulting in UK standards for what they referred to as a "normal" roundabout.

Research and experiences in various developed countries, have slowly reshaped the concept of the older traffic circles, rotaries, gyratories and roundabouts into a more refined form of intersection control, patterned after the UK normal roundabout, which we refer to in this report as the “modern roundabout”.

Thus the evolution of normal or modern roundabouts started in 1966 with a new priority rule and a trend towards smaller and slower roundabouts, progressing through a series of design guidelines culminating in the 1983 UK specifications for the “normal roundabout”.

2.3 Modern Roundabouts in the US

The modern roundabout came to the US in early 1990. The first two US roundabouts were built in Summerlin, Nevada. [Jacquemart, 1998].

Modern roundabouts are beginning to be considered an alternative traffic control device (TCD) that can improve safety and operational efficiency at intersections when compared to other conventional intersection controls. The ‘yield-at-entry’ or ‘off-side priority’ rule at a roundabout assigns priority to the circulating vehicles. They operate like a series of T-intersections. A yield sign is posted at the entry to maintain fluidity and control. All entering vehicles on the approaches have to evaluate a gap in the circulating flow before entering the circulating traffic. Modern roundabouts have deflection for the entering traffic usually in the form of raised islands (splitter islands) and a raised central island. The splitter islands direct traffic towards a central island, which further deflects vehicles to the right. Deflection results in lower speeds and improved safety. Earlier designs treated traffic circles as weaving sections and they were designed with long weaving sections resulting in large circles with high entry and circulating speeds. They could be confusing and unsafe, i.e., many had a high crash risk.

Many modern roundabouts also have flared approaches. The widening of the approach road may allow additional entrance lanes, thus increasing the flexibility of operation for drivers and enhancing capacity. According to the Federal Highway Administration’s roundabout guidelines, (FHWA Guide) ‘Modern roundabouts range in size from mini-roundabouts (outside diameters as small as 50 ft [15m]), to compact roundabouts (outside diameters between 98-115m [30-35ft]), to large multilane roundabouts (up to 492ft or [150m] in diameter) with more than four entry points’ [FHWA, 2000]. However, some experts believe that anything with an outside diameter over 200ft (66m) is too large to be called a modern roundabout [Wallwork, 2000]. In fact Wallwork claims that the roundabouts built in Summerlin, NV in 1990 were too big to be modern roundabouts and he claims he designed the first in Florida in 1991 [Wallwork, 2000]. Irrespective of which actually was the first, the important fact is that any circular intersection built in the US prior to 1990 is not a modern roundabout.

Modern roundabouts are being implemented throughout the US in a variety of situations. Many states and cities are considering roundabouts as a viable alternative to other TCD’s, and, in some cases, complex freeway interchanges. The safety record of well designed modern roundabouts is excellent.

2.4 Roundabout Safety

Single-lane roundabouts may perform better than two-way stop-controlled (TWSC) intersections in the U.S. under some conditions [Flannery & Datta, 1996]. Crash reduction after installation of small and medium capacity roundabouts appears to be less than at large, multilane roundabouts; however, overall crash frequencies are generally reduced and injury crash frequencies are significantly reduced. Crash reductions at modern roundabouts are most pronounced for motor vehicles, less pronounced for pedestrians, and indefinite for bicyclists, depending on the study and bicycle design treatments [IIHS, 2000].

Crash studies in other countries concluded that roundabouts are safer than comparable intersection alternatives. Table 2.1 summarizes the comparison of mean reduction of crashes in different countries.

TABLE 2.1: Mean Crash Reductions in Various Countries

Country	**Mean Crash Reduction (%)**	
	All Crashes	Injury Crashes
Australia	41-61%	45-87%
France		57-78%
Germany	36%	
Netherlands	47%	
United Kingdom		25-39%
United States	37%	51%

Source*: FHWA Roundabout Informational Guide, 2000.*

Flannery reported a preliminary comparison of before and after accident rates was performed for eight sites in the US. [Flannery, 2001]. These sites had been operational for two years or more and had accident data available for the before period (when there was no roundabout at that location). From this study it was found that the safety performance of the intersections studied improved in terms of reduced crash frequency, accident rates, and injury rates after installation of roundabouts. See Table 2.2 for results [Flannery, 2001].

TABLE 2.2: Safety Performance Data for Eight Intersections Converted to Roundabouts in the US

Study site	Per. Period	Accident Frequency/year Before/After	Accident Rate (Acc/MEV) Before/After	Injury Accident Rate (Acc/MEV) Before/After
Palm Beach County, FL	2 yrs	1.5/1.5	0.54/0.54	0.5/0.0
Lisbon, MD	2 yrs	7.5/2.5	2.42/0.81	1.5/0.5
Tallahassee, FL	2 yrs	4.5/1.5	0.69/0.23	0.0/0.0
Fort Walton Beach, FL	2 yrs	8.0/2.0	1.83/0.45	2.0/0.0
Lothian, MD	2 yrs	13.0/4.0	2.37/0.73	4.5/1.5
Washington County, MD	2 yrs	4.5/0.0	1.76/0.0	1.0/0.0
Cecil County, MD	2 yrs	3.0/0.0	1.37/0.0	1.0/0.0
Carroll County, MD	2 yrs	5.3/0.0	1.81/0.24	2.25/0.75

Source: Flannery. A. "Geometric Design and Safety aspects of Roundabouts." *Transportation Research Record 1751,* Transportation Research Board, National Research Council, Washington, DC, 2001.

A study by the Insurance Institute for Highway Safety (IIHS) evaluated the changes in motor vehicle crashes following conversion of 23 intersections from stop sign and traffic signal control to modern roundabouts. A before and after study was conducted using an empirical Bayes procedure. The study estimated highly significant reductions of approximately 40% for all crash severities combined and 80% for all injury crashes. The reduction in number of fatal and incapacitating injury crashes were estimated to be 90% [Persaud, et.al, 2001]

A study done by the Maryland Highway Administration at eight of its modern roundabouts (those built between April 1993 and December 1998) that had been in operation for 5 to 10 years revealed that the average annual accidents for the intersections fell from an average of 4.98 accidents/year in the before period, to an average of 1.8 accidents/year in the after period, a 64% reduction. Accident severity also decreased, as injury accidents have shown a reduction from an annual average of 3.0 injury accidents in the before period to an annual average of 0.5 injury accidents in the after period, a reduction of 83%. Each intersection shows a reduction in both total reported accidents and injury accidents. See Figure 2.2 for aggregate results. [UTM 1999]

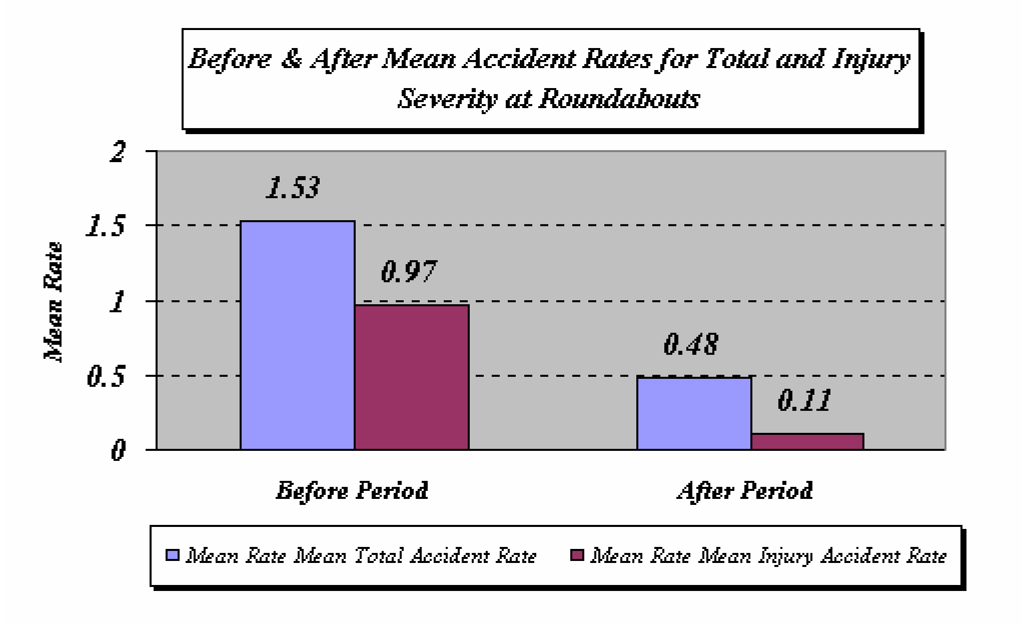


FIGURE 2.2: Figure Showing Aggregate Results for Eight Maryland Roundabout Locations

Basic reasons for the increased safety level at roundabouts are: [FHWA 2000]

- Roundabouts have fewer conflict points in comparison to conventional intersections. The potential for hazardous conflicts, such as right angle and left turn head-on crashes is eliminated with roundabout use. Single-lane approach roundabouts produce greater safety benefits than multilane approaches because of fewer potential conflicts between road users, and because pedestrian crossing distances are short.
- By installing a modern roundabout in place of other conventional intersection traffic control types, conflict points are reduced from 32 to 8, a 75% reduction in conflict points (see Figure 2.3).
- Low absolute speeds associated with roundabouts allow drivers more time to react to potential conflicts, also helping to improve the safety performance of roundabouts.
- Since most road users travel at similar speeds through roundabouts, i.e., have low relative speeds, crash severity can be reduced compared to some traditionally controlled intersections.

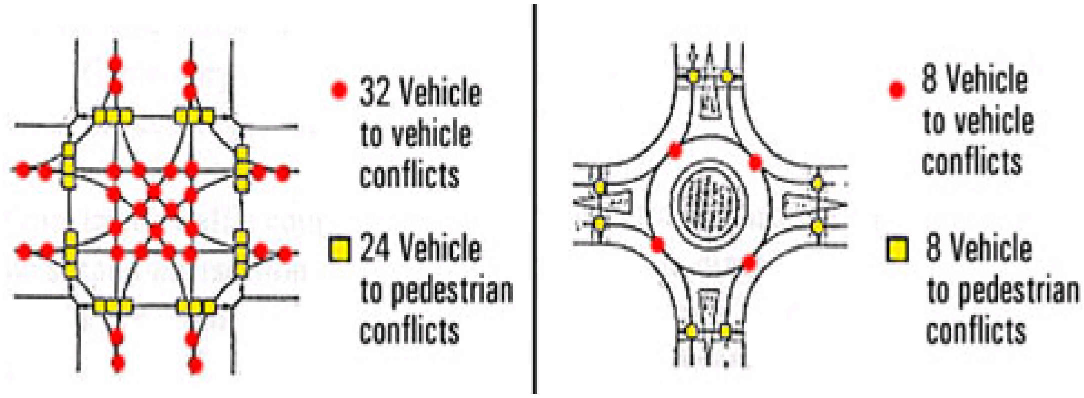


FIGURE 2.3: Figure Showing the Reduction of Conflict Points in a Roundabout When Compared to a Four-Legged Intersection

- Pedestrians need only cross one direction of traffic at a time at each approach as they traverse roundabouts, as compared with un-signalized intersections. The conflict locations between vehicles and pedestrians are generally not affected by the presence of a roundabout, although conflicting vehicles come from a more defined path at roundabouts (and thus pedestrians have fewer places to check for conflicting vehicles). In addition, the speeds of motorists entering and exiting a roundabout are reduced with good design. As with other crossings requiring acceptance of gaps, roundabouts still present visually impaired pedestrians with unique challenges.

Modern roundabouts improve the safety of intersections by reducing potential conflict points, by eliminating or altering crash types and by reducing speed differentials of conflicting movements at intersections, and by forcing drivers to decrease speeds as they proceed into and through the intersection. [FHWA, 2000]

As stated by Jaquemart [1998]:

> "The high capacity and fluidity achieved by the modern roundabout are two main reasons for its success. The substantial reduction in injury accidents has been the primary reason for great success of modern roundabouts in France, Germany, Australia and UK The fact that drivers do not have to wait as long at roundabouts as at signalized intersections makes the roundabouts friendlier to both the driver and to the environment"

2.5 Roundabout Geometry

Modern Roundabout geometric features play a major role in improving safety and operational efficiency of a modern roundabout as an intersection control. [AUSTROADS, 1993; Russell, et. al., 2000; FHWA 2000]. Geometric elements of a roundabout are shown in Figure 2.4.

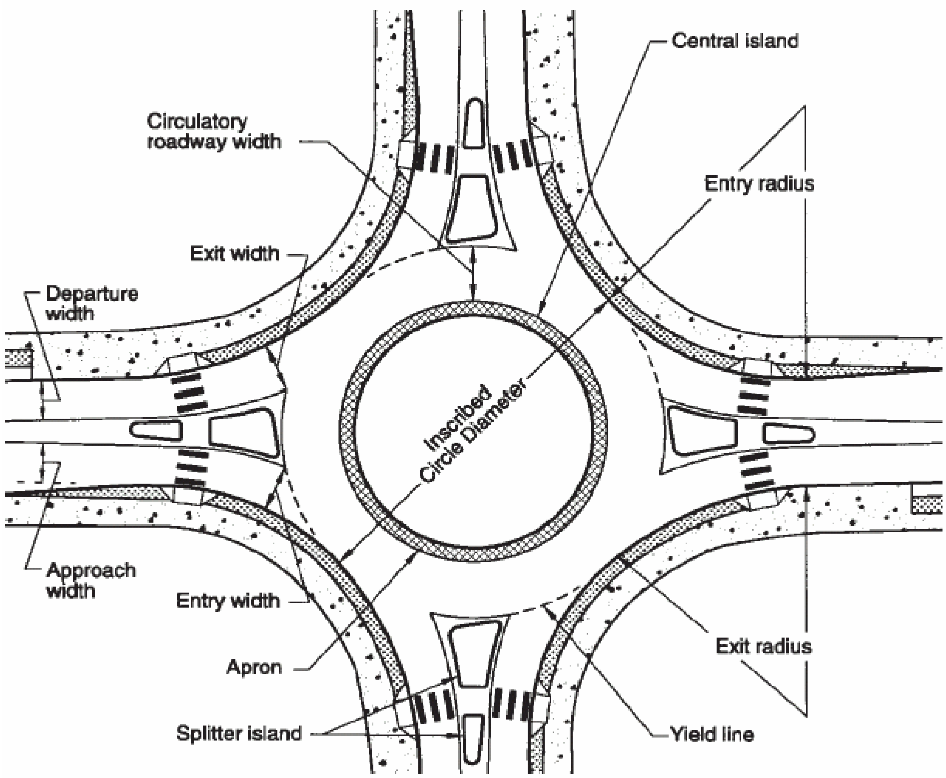


FIGURE 2.4: Geometric Elements of a Modern Roundabout
(Source: FHWA Guide 2000)

The geometric elements are defined as follows in the FHWA guide: [FHWA, 2000]

- **Circulating road width:** The width of the circulating roadway on which the vehicles circulate to reach their preferred exits. It is the width between the outer edge of the roadway and the central island excluding the width of the truck apron.
- **Inscribed diameter:** The diameter measured between the outer edges of the roadway. This includes the circulating roadway, truck apron and the central island.

- **Entry and exit width:** The perpendicular length from the right edge of the entry/exit to the intersection point of the left edge line and the inscribed line.
- **Entry and exit radii:** The minimum radius of curvature of the outside curb.
- **Approach and departure width:** The width of the approach/departure lane used by traffic stream to enter/exit the intersection.

2.6 Roundabout Characteristics

Modern roundabouts have superior operational characteristics (i.e. capacity, delay, queue length, proportion stopped, etc.,). The capability of reducing the frequency of crashes and crash severity makes it safer than other TCDs. [FHWA 2000; IIHS 2000; Russell, et al., 2000; Jacquemart 1998; Garder, 1998; Flannery, 2001; Austroads 1993; Garder et al., 2000; Flannery & Datta 1996; Alcelik and Besley 1998; HWS consultant group 2001].

2.6.1 Other Considerations

Modern roundabouts are becoming popular in the US for more than just safety reasons. As stated in an article by the Insurance Institute for Highway Safety (IIHS) [San Diego Earth Times, May 2001] "They're less expensive than intersections controlled by traffic signals, saving up to $5,000 per year per intersection in electricity and maintenance" [IIHS, 2001].

They also reduce fuel consumption and vehicular emissions by reducing stops and delays at intersections, and they reduce noise levels by making the traffic flow more orderly. Modern roundabouts can enhance the aesthetics of the place and create visual gateways to communities or neighborhoods. In commercial areas they can improve access to adjacent properties. [IIHS, 2001]

2.7 Australian Guidelines

The Australian guide to traffic engineering practice for roundabouts lists some situations at intersections, where modern roundabouts are appropriate and where they are not inappropriate.

Modern roundabouts may be appropriate in the following intersections: [Austroads 1993]

- "Where the traffic volumes on the intersecting roads are such that "Stop" or "Yield" signs or the "T" junction rule (i.e. turning vehicles, give way to all traffic crossing or coming from the right (left in US)) results in unacceptable delays for the minor road traffic. In these situations, roundabouts would decrease delays to minor road traffic, but increase delays to the major road traffic.
- Where there are more than four legs and/or when conventional intersection controls face difficulty in defining priorities and require large numbers of phases in the case of traffic signals.
- Where there are disproportionately high numbers of crashes.
- Where there is a high proportion of right (left in US) turning traffic.
- At rural cross roads (including those in high speed areas) at which there are crash problems involving right angle collisions.
- At locations where traffic growth is likely to be high and future traffic patterns are likely to be uncertain and changeable.
- Where either of the crossroads needs to be given a priority, and,
- Where major roads intersect in "Y" or "T" "

Modern roundabouts may be not inappropriate in the following intersections:

[Austroads 1993]

- "Where a satisfactory geometric design cannot be provided due to insufficient land space or unfavorable landscape or unacceptable high cost in construction.
- Large combination vehicles and over-sized vehicles frequently use the intersection and insufficient space is available to provide the necessary geometric layout.
- Where there are highly unbalanced flows resulting in higher delays on one or more approaches.
- Where a minor and a major road intersect and there is unacceptable delay for the major road traffic. Roundabout causes delay and deflection to all the traffic, whereas

control by two-way stop or yield or the ‘T-junction’ rule would result in delays to only the minor road traffic.

- Where it is isolated in a network of progressive traffic signals.
- Where peak period, reversible lanes may be required, and
- Where traffic flows leaving the roundabout would be interrupted by a downstream traffic control, which could result in back-ups that influence the operation of the modern roundabout.”

2.8 Kansas State University (KSU) Roundabout Studies

- **Modeling Traffic Flows and Conflicts at Roundabouts:** The researchers at KSU conducted this study for Mack Blackwell National Rural Transportation Study Center (MBTC). The study aimed at providing a basis for understanding the operation of modern roundabouts in Kansas [Russell, 2000]. The study first compared the operational performance of a modern roundabout (the first in Kansas) to two comparable two-way stop controlled intersections and two four-way stop controlled intersections using a computer program called Signalized and Unsignalized Intersection Design and Research Aid (SIDRA). Six operational performance measures available in SIDRA; average delay, maximum approach delay, 95th percentile back of queue, proportion stopped, maximum proportion stopped and degree of saturation were used to compare these intersection control alternatives. The study also compared the operational performance of a single-lane modern roundabout to other traditional intersection control that could have been used to replace two-way stop control for the traffic and geometric conditions existing at the study sites; namely, four-way stop with turn lanes. This project was completed in the year 2000 and the report can be downloaded from the MBTC website. [http://www.mackblackwell.org/research]

  The report concluded that the roundabouts performed better than four-way stop control and four-way stop control with turn lanes on all six performance measures and roundabouts performed better than two-way stop control on all measures except for the average vehicle delay. This research project helped to establish that even at relatively low traffic volumes, roundabouts could be more efficient than two-way and four-way stop control as traffic control at an intersection [Russell, 2000].

- **Operational Evaluation of Modern Roundabouts:** The researchers at KSU conducted this study for the Insurance Institute of Highway Safety (IIHS). It compared the operation during the 'before and after' periods at three modern roundabout locations. These locations are at Hartford County, MD, Reno, NV and Hutchinson, KS. All the locations had a two-way stop control in the before condition and a single-lane, modern roundabout in the after condition. The computer program SIDRA was used to evaluate the operational performance of the two intersection control types. The study was completed and a status report was issued by IIHS in July 2001. The report concluded that "installing a roundabout reduced delays by about 20% and the proportion of vehicles stopping by 14% to 37% across all the three sites. [IIHS, 2001]. The study results were published in the July 28th issue of the IIHS newsletter "Status Report" and can be found on their website.
- **Further Studies of Roundabouts:** This was a before and after study of a roundabout in Hutchinson, Kansas, at 23rd Street and Severance Avenue. The main objective of this study was to use the six measures of effectiveness (MOEs) available in SIDRA to compare and evaluate the performance of the intersection for the before condition with two-way stop control (TWSC) and the after condition with a modern roundabout (RA). When compared to the before condition, the after condition had a 51% reduction in the 95th percentile queue length, a 12% reduction in the average intersection delay, a 47% reduction in the maximum approach delay, a 13% reduction in the proportion stopped, a 30% reduction in the maximum proportion stopped and 40% reduction in the degree of saturation. All the reductions were statistically significant except for the average intersection delay. These results indicate that, there was a significant increase in the operational efficiency after the installation of the modern roundabout at 23rd and Severance. Further theoretical analysis showed that the roundabout also operated more efficiently than a traffic signal would have. On analyzing the crash

history for the study site, it was found that there was an 88% reduction in the number of crashes (at the time the study report was written) when comparing one year and three months before and after periods, indicating that the modern roundabout (after condition) is operating safer than the two-way stop control (before condition). The Level Of Service (LOS) was also improved in the after condition. This study provided additional evidence to support the conclusion that modern roundabouts are not only safer but they are also capable of increasing the efficiency and the LOS of an intersection when compared to other conventional traffic control devices. [Russell et.al, 2001, IIHS 2001]. This report, *Further Studies of Roundabouts*, is available from the MBTC website.

- **Exploration of the Effects of Operational and Physical Characteristics on Operating Speeds at Modern Roundabouts:** The modern roundabout has been found to be a safe and effective intersection configuration in the United States. The design of modern roundabouts and their ability to be safe and efficient depends on their low and consistent operating speeds. This research provided an initial exploration into the relationships of thirteen operational and physical characteristics of the modern roundabout and their effect on operating speeds. These thirteen characteristics were used to develop an operating speed prediction model. Operating speed data from fifty-nine approach movements at twelve modern roundabouts was collected and used in model development. The twelve modern roundabouts studied were located in California, Kansas, Maryland, Mississippi, Nevada and Washington. All data was collected during the summer and fall of 2000. Operating speed prediction equations were developed through the multiple regression process. The variables found to influence operating speed in the final model were circulating lane width, deflection of through vehicles, approach speed, entry radius, central island diameter and angle of turn from entry to exit. This speed prediction model should provide designers insight into the factors that will affect operating speed of modern roundabout.
- **Evaluation of the Road Diet Concept and Comparison to the Operational Performance Of a Single-Lane Modern Roundabout and a Traffic Signal:** The term "Road Diet" is a relatively recent term used to mean a reduction in the

number of travel lanes, usually from four to three. The intersection studied in this project is the intersection of 44th Avenue and 67th Avenue, in University Place, Washington. The existing traffic control was two-way stop. There were three parts to this study. The first ws to analyze the effect of conversion from four lanes to three (road Diet). The second part was to theoretically analyze the intersection assuming a traffic signal had been installed instead of the Road Diet. The third part was to theoretically analyze the intersection assuming a modern roundabout had been constructed instead of the road Diet. The operation of the roadways at the intersection was videotaped and the traffic flow data collected was extracted from these tapes and analyzed using SIDRA (Signalized and Un-signalized Intersection Design and Research Aid) software. Six measures of effectiveness (MOEs) were obtained and compared for three conditions: Road Diet with TWSC, original, four lanes with with a signal and a modern roundabut. All the MOEs (Average queue Length, Proportion of vehicles stopping at intersection, Maximum proportion of vehicles stopping on an approach, Maximum intersection delay, Maximum approach delay and degree of saturation) were statistically compared to determine which roadway configuration and intersection control performed better. It was found that the three-lane roadway configuration reduced the conflict rate and performed better than or equal to the four-lane roadway configuration. Thus, was concluded that three-lane roadway configurations (Road Diet) can be used as a viable alternative for four-lane roadway configurations. Additionally it was concluded that a single-lane modern roundabout would have been the most efficient form of intersection control at this intersection studied, based on the six MOE's. This study was done for IIHS and was never published. Papers available are:

  - Eugene R. Russell, **Srinivas Mandavilli**, "Analysis of a Road Diet Conversion and Alternative Traffic Controls", ITE Technical Compendium of Papers 2003.

- **Environmental Impact of Kansas Roundabouts:** Problems posed by the environmental impacts of traffic are growing and are posing a challenge to traffic engineers. Modern roundabouts can improve traffic flow as well as cut down vehicular emissions and fuel consumption by reducing the vehicle idle time at intersections and thereby creating a positive impact on the environment. The primary objective of this research was to study the impact of modern roundabouts in Kansas in reducing vehicular emissions. Three cities in Kansas; (namely, Olathe, Lawrence, and Paola, where a modern roundabout has replaced a stop controlled intersection) have been chosen for the study. The operation of the roadways at the intersection was videotaped and traffic flow data was extracted from these tapes and analyzed using aaSIDRA (Signalized and Un-signalized Intersection Design and Research Aid) software, version 2.0. The software produces many Measures of Effectiveness (MOEs) of which four were chosen for analyzing the environmental impact of roundabouts. The chosen four MOEs give rate of emission of HC, CO, NOX, and CO_2 in (kg/hr). All the MOEs were statistically compared to determine which intersection control performed better. After observing the MOEs at all locations for the before and after traffic volumes, it was found that the modern roundabout performed better than the existing intersection control (i.e. stop signs) in cutting down vehicular emissions, thereby resulting in a positive impact on the environment. The research concludes that a modern roundabout can be considered a viable alternative to cut down vehicular emissions and thereby making intersections more environmentally friendly. This study resulted in the following papers:

- **Srinivas Mandavilli**, Eugene R Russell, Margaret Rys (speaker), "Environmental Impact of Kansas Roundabouts", 8th Annual International Conference on Industrial Engineering Theory, Las Vegas, Nevada, November 2003.
- **Srinivas Mandavilli**, Eugene R Russell, Margaret Rys, "Modern Roundabouts in United States -An Efficient Intersection Alternative for Reducing Vehicular Emissions", Transportation Research Board, National Research Council, Washington, D.C., January 2004. (Poster Session)
- **Srinivas Mandavilli**, Eugene R Russell (speaker), Margaret Rys, "Environmental Impact of Kansas Roundabouts", Transportation Annual Conference of the Transportation Association of Canada, September 2003.

- **Srinivas Mandavilli**, Eugene R Russell (speaker), Margaret Rys, “Impact of Modern Roundabouts on Vehicular Emissions”, Mid-Continent Transportation Symposium, Iowa State University, Ames, Iowa, August 2003

Another study conducted by the researchers at KSU for IIHS, studied the before and after performances at three intersections in Kansas, Maryland, and Nevada which were controlled by TWSC before being converted to modern roundabouts [IIHS 2001]. The study concluded that the modern roundabouts at these three locations perform better than the TWSC they replaced, by reducing the average intersection delay by about 20% in each case and reducing the proportion of vehicles having to stop by 14% to 37% at the three sites [IIHS 2001].

2.8 Summary

The material reviewed supports a conclusion that a modern roundabout substantially reduces the conflict points and increases both safety and operational efficiency when compared to other, conventional intersection traffic control devices.

Chapter 3

Software Selection

3.1 General

This chapter gives a description of the **S**ignalized & unsignalized **I**ntersection **D**esign and **R**esearch **A**id (SIDRA) software, which was used for the analysis. The KSU research team decided to use SIDRA primarily because of its convenience in comparing key parameters related to operational efficiency at all types of intersection traffic control. Other available roundabout software (e.g. RODEL and ARCADY) do not analyze conventional traffic control. The following sections are taken from earlier studies conducted by KSU researchers. [Russell et.al, 2000, Russell et.al, 2002, Mandavilli, 2002]

3.2 SIDRA Software

The software that was used for data analysis is a.a.SIDRA, Version 1.0. The Australian Road Research Board (ARRB), Transport Research Ltd., developed the SIDRA package as an aid for design and evaluation of intersections such as signalized intersections; roundabouts, two-way stop control, and yield-sign control intersections.

In a modern roundabout performance evaluation by Sisiopiku and Un-Oh (2001) used SIDRA, they found that:

> *"SIDRA provides the same level of service (LOS) criteria for roundabouts and traffic signals under the assumption that the performance of roundabouts is expected to be close to that of traffic signals for a wide range of flow conditions."* [Sisiopiku et.al, 2001].

The input to the SIDRA software includes the road geometry, traffic counts, turning movements, and speed of the vehicles. SIDRA relies upon peak flow period and the peak flow factor or the peak hour factor (PHF). These parameters have a large effect on the overall results when varying them [Alcelik and Besley 1998]. These are user specific and the software gives flexibility to adjust for local conditions. The flow period for this study was fixed at 60 minutes, with a peak flow period rate of 15 minutes. The PHF was calculated for every 6-hour period by the equation 3.1 as defined in HCM 2000.

$$PHF = \text{Peak hour Volume} / (4 * \text{Maximum (Peak flow period volume)}) \quad \text{..........(3.1)}$$

The PHF varied depending on the peak hours and the 15-minute peaks collected over the period. The raw data collected represents the field data but SIDRA will modify those counts based on the PHF from the equation 3.1. The modified volume is given by the equation 3.2 below: [Alcelik and Besley 1998]

$$\text{Volume}_{(SIDRA)} = \text{Volume}_{(Field)} / PHF \quad \text{..........(3.2)}$$

The SIDRA software analyzes the data and the output provides measures of effectiveness from which the performance of the intersection can be determined. For analyzing a modern roundabout the software uses the theory of gap acceptance in predicting the performance measures of effectiveness (MOEs). Based on the turning movements and the geometric parameters, SIDRA output provides the MOEs to evaluate various intersection types. It predicts 19 MOEs for all the intersection control type. They are: [Russell et.al,2001]

- intersection level of service,
- worst movement level of service,
- average intersection delay (s),
- maximum average movement delay (s),
- largest back of queue (ft),
- degree of saturation-highest among the lane group (%),

- practical spare capacity-lowest among the lane group (%),
- total vehicle capacity –all lanes (veh/h),
- total vehicle flow (veh/h),
- total person flow (pers/h),
- total vehicle delay (veh-h/h),
- total person delay (per-h/h),
- total effective vehicle stops (veh/h),
- total effective person stops (pers/h),
- total vehicle travel (veh-mi/h),
- total cost (US$/h),
- total fuel (ga/h),
- total CO_2 (kg/h) and
- total lead emission (kg/h).

Even though there are 19 measures of effectiveness (MOEs) given by SIDRA output, only six of them were considered relevant to this project.

Based on the Level of Service (LOS) concept, the measures of effectiveness should include the degree of saturation (v/c) ratio and delay. The US Highway Capacity Manual (HCM) recommends using delay for all intersection alternatives. For signalized intersection control, it recommends analyzing the delay and capacity simultaneously to evaluate the overall operation. Hence the Average Intersection Delay, Maximum Approach Delay and Degree of Saturation were MOEs chosen in previous studies [Sisiopiku et.al, 2001].

According to McShane and Roess "Length of queue at any given time is a useful measure and is critical in determining when a given intersection will begin to impede the discharge from an adjacent upstream intersection" [McShane et.al, 1998]. Hence the Average Queue Length was chosen as another MOE. The proportion of vehicles stopping at an intersection is related to the queues that form and to delays that occur at the intersection. Therefore, the Proportion Of Vehicles Stopped, and Maximum Proportion Of Vehicles Stopped were also chosen as MOEs.

These six measures of effectiveness discussed above were chosen because the authors believe they directly relate to the operational effects of the roadway. The other SIDRA measures are more related to environmental effects, and were not considered. [Russell et.al, 2000, Russell et.al, 2002, Mandavilli, 2002]

To summarize, the six measures of effectiveness used in this study to evaluate performance are: [Russell et.al, 2000, Russell et.al, 2002, Mandavilli, 2002]

1. 95th Percentile Queue Length,
2. Degree Of Saturation,
3. Average Intersection Delay,
4. Maximum approach Delay,
5. Proportion Of Vehicles Stopped, and
6. Maximum Proportion Of Vehicles Stopped.

These are defined below: [Alcelik and Besley 1998]

- **95th Percentile Queue Length**: SIDRA gives a percentile queue length in the output. This is defined as: "A percentile queue length is a value below which the specified percentage of the average queue values observed for individual cycles fall." [Alcelik and Besley 1998]. 95th percentile queue length value was used in analysis.
- **Degree of Saturation:** This measure gives us a measure of the congestion on the roadway that is being used by the traffic. It is the ratio of volume to capacity. Here the volume of the vehicles is input and the capacity is calculated by SIDRA.
- **Average Intersection Delay:** This measure gives the average vehicle delay for all the vehicles entering the intersection.
- **Maximum Approach Delay:** This measure gives the average vehicle delay for the approach with the highest average delay. As stated by the a.a SIDRA manual:

"Delay to a vehicle is the difference between interrupted and uninterrupted travel times through the intersection. SIDRA delay estimates are based on the path-trace method of measuring delays. This includes all delays experienced by vehicles arriving during the demand flow period even if some of those vehicles depart after the analysis period. Both interrupted and uninterrupted travel times measured by an instrumented car include the intersection geometric delay, hence the delay measured by this method is the stop-line delay (equal to the queuing delay + major stop-start delay)" [Alcelik and Besley 1998].

- **Proportion of Vehicles Stopped:** This measure gives the proportion of vehicles that are approaching the intersection and are required to stop due to the vehicles already present in the intersection.
- **Maximum Proportion of Vehicles Stopped:** This measure gives the highest proportion of vehicles that are stopped on one approach due to the vehicles already present in the intersection.

Many engineers who design or analyze modern roundabouts believe in and rely on the output of SIDRA. However, it should be noted that there are other engineers who believe that gap acceptance theory (basis of SIDRA) does not accurately predict roundabout capacity for high-volume roundabouts. [Crown 2003, McCullough 2003].

They are of the opinion that empirically based programs such as RODEL and ARCADY are more accurate predictors. Conclusions regarding either approach are beyond the scope of this study. The KSU research team believes that:

1. The results of any of these programs should not be significantly different in the mid ranges of traffic volume that exist at the roundabouts in this study and
2. Since the study looks at before and after differences in the results, if the program output were low or high it would likely be in the same direction and magnitude in both cases, diminishing the effect on the difference.

Chapter 4

Data Collection

4.1 General

The methodology used for the data collection is similar to the one that was adopted in earlier studies by KSU researchers. Unless specifically noted otherwise, the following sections are taken from those studies. [Russell et.al, 2000, Russell et.al, 2002, Mandavilli, 2002, Sathya 2002]

4.2 Data Collection

The data collection consisted of two phases. The first phase was data collection in the field using a camera and video recorder. Tapes and the second phase was obtaining traffic counts visually from the videotapes that recorded the field data.

4.2.1 Phase 1: Video Data Collection

The benefit of using this method for data collection is that all the data is recorded on videotapes and can be accessed and retrieved at a later time. Also, the tapes serve as a permanent record for verification of data. A specially designed 360°-omni directional, video camera and videocassette recorder were used for data collection at each location. The camera was designed by Intelligent Highway systems, Inc., (White Plains, NY). The camera was designed to provide a full 360° view when mounted above the intersection.

FIGURE 4.1: Camera Mounted on a Lamp Pole
(Photo courtesy: Dr. Russell)

FIGURE 4.2: TV/VCR Used for Recording
(Photo courtesy: Dr. Russell)

FIGURE 4.3: VCR/TV Signal Steel Cabinet
(Photo courtesy: Dr. Russell)

The camera was placed near the intersection to see the traffic flow coming toward and leaving the intersection on all legs simultaneously. The cameras were installed on existing poles and mounted perpendicular to the ground. The perpendicular mounting allowed the video image to be relatively distortion free to the horizon in all directions. The camera was mounted approximately 6 meters (20 feet) above the ground. This mounting height provides a focal plane of approximately 40.5 meters by 54.0 meters (133 feet by 177 feet). The camera feed went in to a

TV/VCR unit placed in a recycled traffic signal controller cabinet. All the equipment was mounted on a single pole. The video images were recorded on standard VHS videotapes. [Mandavilli, 2002]

Data from the intersection was collected in the before condition (when the intersection was controlled by stop signs) and in the after condition (after a modern roundabout was built at the intersection). The traffic counts from the intersection were video taped for two six-hour sessions from 7:00AM-1:00PM and from 1:00PM-7:00PM on normal week days for the before and after conditions. A normal day in this study refers to a day with no adverse environmental/weather or any external factor(s), such as special events in the nearby locality of the study intersection that would impact the flow of traffic through the study intersection.

4.2.2 Phase 2: Visual Data Collection

In this phase the data was visually collected from the videotapes. All the videotapes were studied visually to extract the traffic volumes and turning movements for the analysis. Various student graduate research assistants in the Department of Civil Engineering at KSU did the data extraction from the videotapes. [Russell et.al, 2001]

Every vehicle coming from all the approaches for a period of fifteen (15) minutes was recorded on pre-prepared data collection sheets (see Figure 4.4). The right turning movements were marked on the right-hand side box (R), through movements (T) on the middle box and left turning movements (L) on the left-hand side box, respectively, for all the approaches. [Russell et.al, 2001]

For one study, an attempt was made to record conflicts, i.e. a conflict analysis. There were too few conflicts to make any meaningful conclusions, so the effort was dropped. [Mandavilli, 2002]

Subsequently data from these sheets were entered in spreadsheets (MS-Excel) to calculate the hourly volumes and peak hour factors (see Figure 4.5). Hourly counts were used as input data for analysis using the computer program aaSIDRA (Signalized and Un-signalized Intersection Design and Research Aid). The tapes were also watched for conflicts for each fifteen-minute interval.

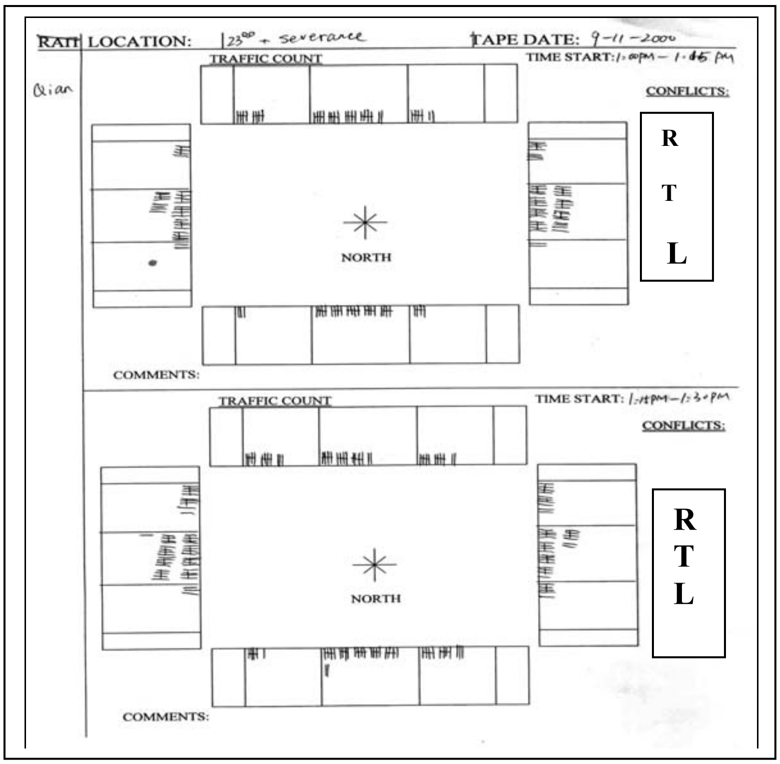


FIGURE 4.4: Pre-Prepared Volume Counts Mark Sheet
(Source: Sathya, 2000)

Location: **HUTCH BEFORE** **Date:**

Time Start:	North Approach Right	Thru	Left	Total	East Approach Right	Thru	Left	Total	South Approach Right	Thru	Left	Total	West Approach Right	Thru	Left
1:00 PM	10	54	14	78	11	37	13	61	10	37	6	53	6	59	5
1:15 PM	9	39	10	58	8	62	10	80	10	38	7	55	3	35	0
1:30 PM	9	25	6	40	3	27	4	34	19	26	4	49	2	42	3
1:45 PM	5	34	8	47	11	51	8	70	14	33	3	50	8	48	5
2:00 PM	8	40	4	52	8	34	8	50	8	28	5	41	2	37	8
2:15 PM	9	42	10	61	4	33	13	50	7	30	7	44	7	35	5
2:30 PM	12	25	13	50	6	45	10	61	6	29	1	36	3	29	6
2:45 PM	6	34	9	49	2	44	8	54	8	29	13	50	6	36	4
3:00 PM	5	35	6	46	9	55	5	69	8	29	11	48	10	43	3
3:15 PM	10	35	10	55	13	54	15	82	6	58	5	69	6	58	5
3:30 PM	7	34	7	48	10	57	8	75	9	37	16	62	10	55	7
3:45 PM	7	40	8	55	11	64	5	80	14	34	7	55	3	45	8
4:00 PM	5	33	5	43	6	50	13	69	9	51	8	68	5	39	7
4:15 PM	3	52	9	64	9	61	9	79	9	49	7	65	4	45	6
4:30 PM	**8**	**46**	**8**	**62**	**12**	**82**	**6**	**100**	**9**	**40**	**3**	**52**	**3**	**46**	**11**
4:45 PM	**7**	**48**	**13**	**68**	**6**	**89**	**11**	**106**	**14**	**38**	**5**	**57**	**3**	**61**	**8**
5:00 PM	**10**	**40**	**8**	**58**	**17**	**79**	**7**	**103**	**10**	**65**	**7**	**82**	**4**	**65**	**10**
5:15 PM	**8**	**39**	**8**	**55**	**15**	**83**	**10**	**108**	**7**	**55**	**12**	**74**	**7**	**43**	**9**
5:30 PM	7	34	8	49	13	59	16	88	4	48	5	57	10	39	12
5:45 PM	9	41	9	59	11	56	5	72	5	31	8	44	4	39	7
6:00 PM	3	28	18	49	9	39	7	55	9	43	4	56	10	39	8
6:15 PM	6	34	4	44	5	40	7	52	5	44	5	54	5	38	8
6:30 PM	5	38	6	49	4	31	10	45	6	34	11	51	11	47	9
6:45 PM	6	31	11	48	6	26	3	35	6	23	9	38	6	30	5

Peak Hour:	North Approach Right	Thru	Left	Total	East Approach Right	Thru	Left	Total	South Approach Right	Thru	Left	Total	West Approach Right	Thru	Left
4:30 PM-5:30 PM	33	173	37	243	50	333	34	417	40	198	27	265	17	215	38

Turning Movements:

Time: 4:30 PM-5:30 PM

```
                                North

                        33      173     37
                        <--'    |       '-->
            38    ---^          V               ^---    50
West        215   --->                          <---    333     East
            17    ---v          ^               v---    34
                        <--,    |       ,-->
                        27      198     40

                                South
```

FIGURE 4.5: Excel Spreadsheet: Summary of Visual Data Extracted from Videotapes
(Source: MBTC, 2000)

Chapter 5

Data Analysis

5.1 General

The methodology used for the data analysis is similar to the one that was adopted in earlier studies by KSU researchers. The following sections in this chapter are taken from those studies. [Russell et.al, 2000, Russell et.al, 2002, Mandavilli 2002, MBTC 2002]

5.2 Data Analysis- Standard Before/After Situation

In the typical or standard case where before and after data was available, the data collected from videotapes for the AM and PM periods was recorded manually in 15-minute periods, and hourly data was then input into the SIDRA software for analysis. The SIDRA output was analyzed to obtain the operational performance of the intersection for the different traffic control devices in the before and after conditions. (e.g: Stop Signs before and Modern Roundabout after)

All the six Measures of Effectiveness (MOEs) used were statistically compared using the standard statistical procedures as described below in this report. The data analysis was done separately for the AM and PM hourly volumes but the procedure followed was the same for both sets of data. This was done to see whether the results differed due to the differences in before and after traffic volumes for the AM and PM traffic counts, as there may have been more traffic during the PM period or during the AM period.

5.2.1 Traffic Volumes

When the traffic volumes were collected from the tapes, if it were found statistically that the before and after traffic volumes differed significantly for either the AM or the PM periods, then they were adjusted to make the before and after traffic counts statistically similar. To do this, hourly traffic counts were viewed and subjectively eliminated from the higher set until the two sets tested "statistically similar". This procedure was adopted so that the roadway conditions being compared would not be biased due to the effect of differing traffic volumes. In the elimination process, usually one high and one low count were eliminated. The statistical techniques used are discussed in detail below in the Statistical Analysis section of this report.

5.2.2 Statistical Analysis

Before going to the SIDRA analysis a statistical analysis techniques were used to test whether the before and after traffic volumes for the before and after (Roundabout) intersection control conditions were statistically similar. Since a comparison is being made between two different intersection controls, it is essential that the traffic conditions are similar for both conditions; else the comparison made may not be a valid comparison. [Russell et.al, 2000].

Statistical tests were performed for the evaluation of the operational performance of a modern roundabout compared traditional intersection control existing "before" using Statistical Analysis Software (SAS version 6.0) available on KSU UNIX Computer System. First the base assumptions of Normality and Equal Variances were tested for the data sets in order to determine the specific type of statistical test to be used in evaluating the intersection operation using the six SIDRA MOEs described previously.

Following is a description of the typical statistical tests run on the comparisons made in this study.

The first test is the Normality test. The normality of the data set is determined based on the inter quartile range/standard deviation (IQR/S) value and Shapiro Wilk test. The inter quartile range (IQR) is the difference between the first and the third quartile of the data set (i.e 25^{th} and the 75^{th} percentile values) and is calculated with SAS software. "S" is the standard deviation of the data set, which is also calculated using SAS software. In the first test (IQR/S value) a normal distribution was indicated if the ratio of these two values was near 1.3. This normality indicator is satisfied if the IQR/S was within +/- 50% of the desired value of 1.3. The second test for Normality, the Shapiro-Wilk test, is a sensitive test for smaller data sets and hence an alpha value of 0.01 was chosen to lessen the possibility of false rejection. The test is rejected if the p value is less than the value of alpha (0.01). [Russell et.al, 2000]

The second test is the Equality of Variances test. The equality of variances is tested using Levene's test. This test is sensitive to normality assumptions and hence an alpha value of 0.01 was chosen for the test. If the p value is found to be less than the alpha value, the test is rejected.

Based on the results of the Normality and the Equality of Variances tests, further tests are conducted. If the sample is found to be Normal and satisfied Equality of Variances, then the equality of the means is tested using the analysis of variance (ANOVA), F-Test. An alpha value of 0.05 is used for this test. If the p value was found to be less than alpha value then the statistical process is ended. Failure to reject the null hypothesis meant that the means were considered to be statistically equal. If a rejection of the null hypothesis is made then the means are considered to be unequal. If a rejection is made, then the Tukey's and Duncan's tests would be used to make a multiple comparison to find out which of the means were statistically different. [Russell et.al, 2000]

If the data is found to be normally distributed but has Unequal Variances, the equality of the means is tested using the Welch's test. An alpha value of 0.05 is used for the test. Failure to reject the null hypothesis meant that the means are considered to be statistically equal. If a rejection is made then, the Fischer, Least Difference Test is used to determine which means are statistically different. [Russell et.al, 2000]

If the data is not normally distributed, the Kruskal-Wallis test is used to test whether the data populations were the same or not. An alpha value of 0.05 is used for this test. Failure to reject the null hypothesis means that the means are considered to be statistically equal and the statistical process ends. [Russell et.al, 2000]. The Table 5.1 presents a summary of the statistical tests. [Russell et.al, 2000]

TABLE 5.1: Summary of Statistical Tests

Statistical Test	Inference
NORMALITY TEST	
a. – IQR/S ≈ 1.3.	Sample is normally distributed if ≈ 1.3.
b. – Shapiro Wilk P-Value	H_o: "Sample is normally distributed", α=0.01
EQUAL VARIANCES	
Levene's Test	H_o: $\sigma^2_{AWSC} = \sigma^2_{R.A}$, α=0.01
NORMAL W/EQUAL VARIANCES	
Analysis Of Variance (ANOVA) F-Test	H_o: $\mu_{AWSC} = \mu_{R.A}$, α=0.05
	-Fail to reject H_o, Analysis Stops.
	-Reject H_o, Perform Multiple Comparisons (Tukey's and Duncan's Tests)
NORMAL W/UNEQUAL VARIANCES	
Welch's Test	H_o: $\mu_{AWSC} = \mu_{R.A}$, α=0.05
	-Fail to reject H_o, Analysis Stops.
	-Reject H_o, Perform Multiple Comparisons (Fisher Least Difference Test)
NOT NORMAL	
Kruskal-Wallis Test	H_o: Population distributions are same, α=0.05
	-Fail to reject H_o, Analysis Stops.
	-Reject H_o, Observe data plots to determine rank order.

IQR: Inter Quartile Range, S: Standard Deviation.

Source: "Russell.E.R., Rys M.J., and Luttrell.G., Modeling Traffic Flows and Conflicts at Roundabouts, Mack-Blackwell Report." [MBTC, 2000]

During the process of visual data extraction from the videotapes, it was observed that pedestrian and bicyclists' traffic was low, and they were ignored in the analysis. Heavy vehicle traffic going through the intersection was also light and, was not counted separately. Instead, for purpose of analysis, heavy vehicle traffic was assumed to be 3% of the total traffic volumes on each of the approaches. This process was followed for all sites.

Chapter 6

Description of Study Intersection Sites

6.1 General

This chapter covers the description of the study intersection sites, a summary of the data collected at each intersection site and the traffic volume trends at each study intersection (STIT) site.

6.2 Site Descriptions

6.2.1 Olathe

Two sites were studied in Olathe, the intersection of the Ridgeview Road and Sheridan Avenue and the intersection of Rogers Road and Sheridan Avenue. Sheridan Avenue runs in the East-West direction while the Ridgeview and Rogers roads run in the North-South direction, roughly parallel to Interstate 35 (I-35). Figure 6.1 shows the locations.

1. **OLATHE: Location A**: Intersection of the Ridgeview Road and Sheridan Avenue.

- **Hourly Volumes**: The traffic volume data was collected from 7:00AM to 9:00AM and from 4:00PM to 6:00PM on normal week days for the before and after conditions. The before condition was with the intersection operating on normal days with All -Way Stop Control (AWSC). The after condition was with the intersection operating on normal days after the modern roundabout (RA) was in operation.

  The turning movement counts were obtained for every 15-minute interval and recorded. Periods that had traffic less than 200 vehicles per hour were ignored in the analysis, as it was desired to study their operational performance under higher volumes.

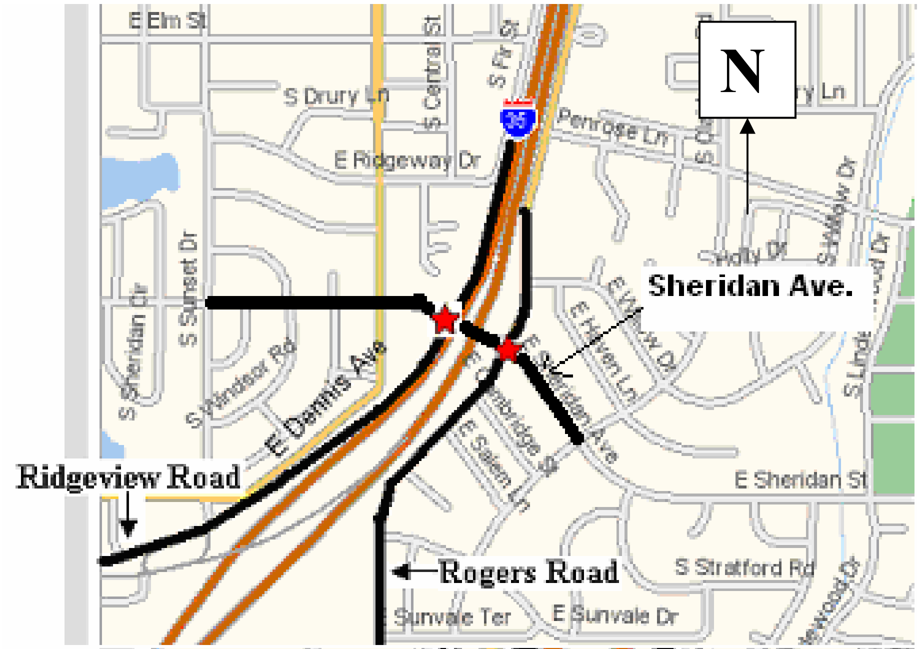


FIGURE 6.1: Figure Showing the Geographic Locations of the Two Olathe Roundabouts
Source: Adopted from Maps.Yahoo.com

- **Before Condition:** Prior to the installation of the modern roundabout at this site the intersection was controlled by stop signs on all approaches (All Way Stop Control-AWSC). All vehicles traversing through this intersection were required to stop before entering the intersection. The major drawback of this type of intersection control is that the presence of vehicles on all the approaches of an AWSC intersection will result in longer departure headways and longer driver decision times that reduce the capacity of the intersection.
- **Geometric parameters - Before condition:** In the before condition there was one approach lane and one exit lane in each of the approaches. The lanes were 12 ft (3.7m) wide. The terrain was flat with zero gradients on all the approaches. See Figure 6.3 for hourly turning movements.
- **After Condition:** In the after condition a modern roundabout was built. This roundabout is a single lane roundabout with a circular central island. Key dimensions are given in the following section.

- **Geometric parameters - After condition:** The roundabout has a circulating lane width (W_c) of 22ft (6.6 m) on a flat terrain (zero gradient on all the approaches). For the North approach and West approaches there are one entry and one exit lanes. The lane width is 12 ft (3.7m). For the South approach and East approaches there are two entry lanes and one exit lane. The lane width is 12 ft (3.7m). The inscribed diameter (D_i) is 40ft (12m). The posted speed limits on the approaches were 20mph (32km/hr). See Figure 6.4 for a detailed drawing of roundabout. See Figure 6.5 for hourly turning movements.

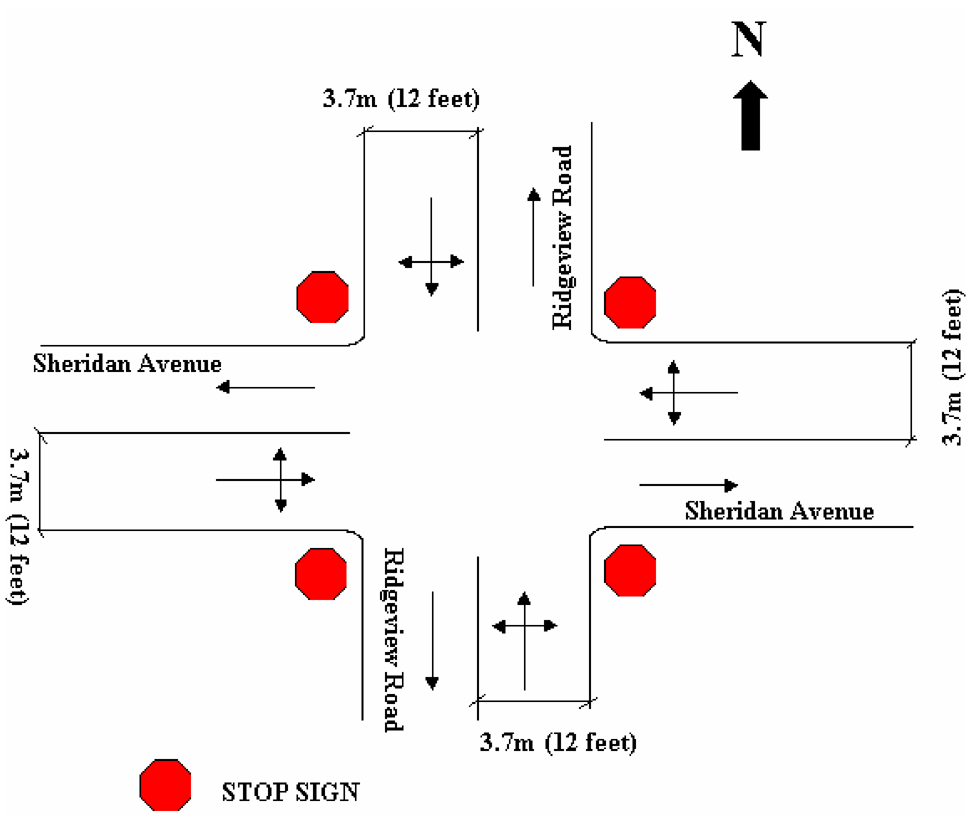


FIGURE 6.2: Ridgeview/Sheridan Intersection - Before Condition

AM Condition
North

```
                          7        47       147
                        <--'        |       '-->
      15       ---^                 V                 ^---      256
West  46       --->                                   <---       58    East
       1       ---v                 ^                 v---      131
                        <--,        |       ,-->
                          1        57       123
```

South

PM Condition
North

```
                         18        81       313
                        <--'        |       '-->
      13       ---^                 V                 ^---      279
West  94       --->                                   <---       73    East
       1       ---v                 ^                 v---      165
                        <--,        |       ,-->
                          1        85       262
```

South

FIGURE 6.3: Hourly Turning Movements for Ridgeview Road and Sheridan Avenue in Before Condition

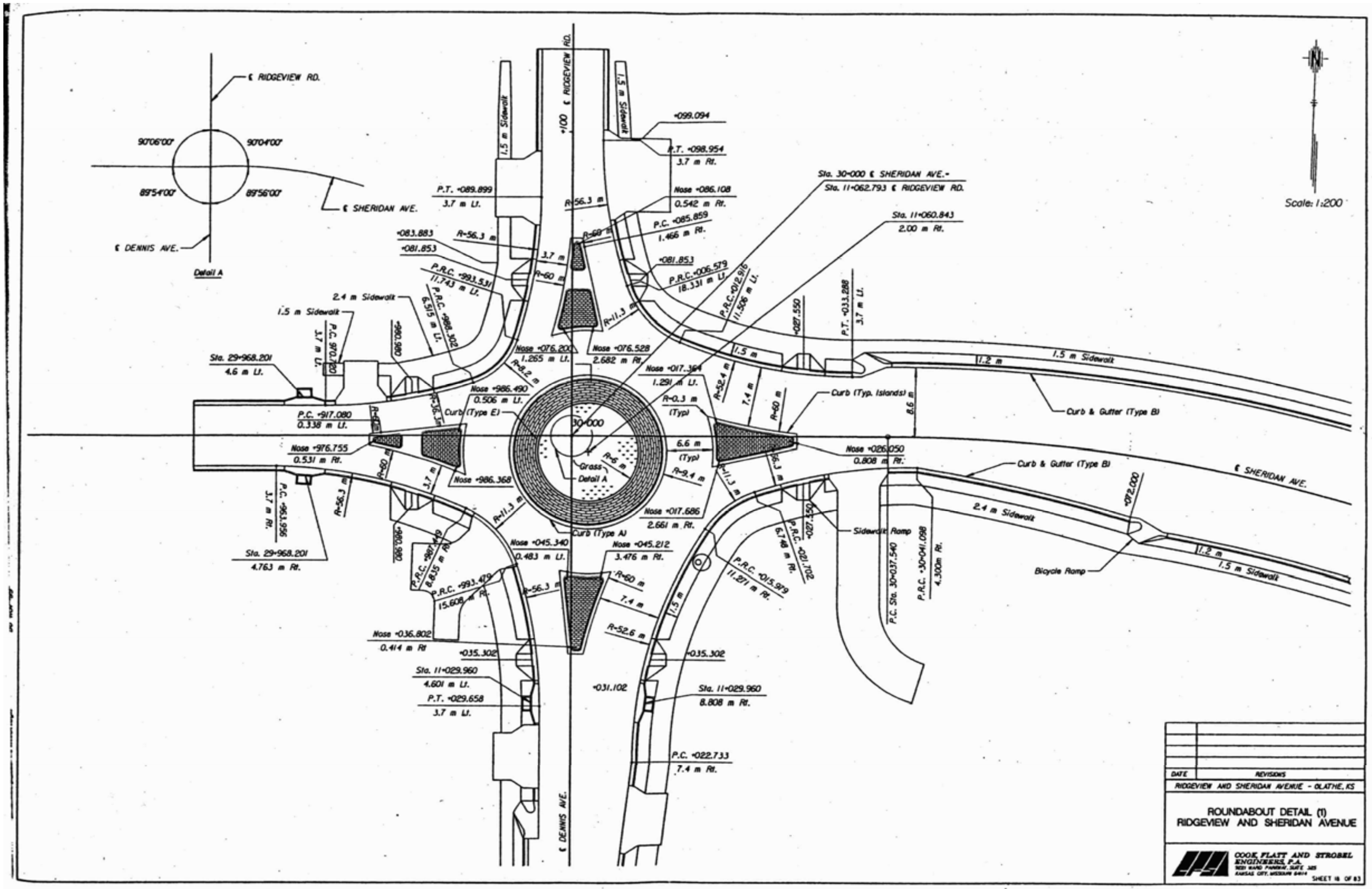


FIGURE 6.4: Ridgeview/Sheridan Intersection - After Condition
(Reduced from actual plan sheet provided by KDOT)

```
                    AM Condition
                        North

              9          77          171
             <--'         |          '-->
   20    ---^             V                  ^---    288
West 27  --->                                <---     25   East
   15    ---v             ^                  v---    110
             <--,         |          ,-->
              6          72          127

                        South
```

```
                    PM Condition
                        North
                  15        132        367

              <--'        |        '-->
   11    ---^             V                  ^---    275
West 43  --->                                <---     44   East
   12    ---v             ^                  v---    139
              <--,        |        ,-->
              13         100       274

                        South
```

FIGURE 6.5: Hourly Turning Movements for Ridgeview Road and Sheridan Avenue in After Condition

- **Statistics for AM and PM periods - Ridgeview Road and Sheridan Avenue:** Tables 6.1 and 6.2 give the statistics for the observed AM and PM period hourly traffic volumes, respectively, with the intersection operating under before (AWSC) and after (RA) conditions and Figure 6.6 shows the traffic volume variation on a typical day with the intersection operating under each conditions.

TABLE 6.1: Descriptive Statistics for the AM Period Observed Hourly Traffic Volumes

Statistics	*Traffic Volumes Veh/hr**	
	Intersection Treatments	
	AWSC	*RA*
Min	708	776
Mean	907	949
Max	1110	1124
Stdev	138	114
No. of Data points	41	31

*** Total Entering Vehicles**

TABLE 6.2: Descriptive Statistics for the PM Period Observed Hourly Traffic Volumes

Statistics	*Traffic Volumes Veh/hr**	
	Intersection Treatments	
	AWSC	*RA*
Min	1040	1321
Mean	1377	1425
Max	1626	1784
Stdev	130	174
No.of Data points	50	40

*** Total Entering Vehicles**

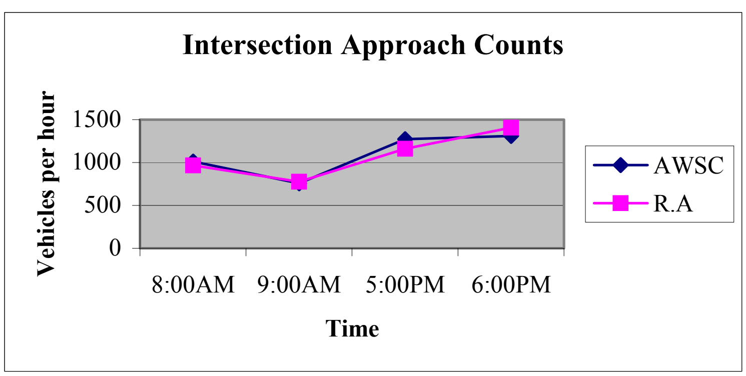


FIGURE 6.6: Traffic Volume Variation for a Typical Day

(Note: Lines in the figure are provided for reading convenience. No conclusions should be made that the lines indicate a statistical distribution or that there is a straight-line relationship between STIT time and vehicles per hour)

2. **<u>OLATHE: Location B</u>:** Intersection of Rogers Road and Sheridan Avenue

- See Figure 6.1 for geographic location of the site.
- **Before Condition:** Prior to the installation of the modern roundabout at this site the intersection was controlled by stop signs on all approaches (AWSC).
- **Geometric parameters - Before condition:** In the before condition there was one approach lane and one exit lane in each of the approaches. The lanes were 12ft (3.7m) wide. The terrain was flat with zero gradients on all the approaches. See Figure 6.8 for hourly turning movements.
- **After Condition:** In the after condition a modern roundabout was built. This roundabout is a single lane roundabout with a circular central island. Key dimensions are given in the following section.
- **Geometric parameters - After condition:** The roundabout has a circulating lane width (W_c) of 24 ft (7.4 m) on a flat terrain (zero gradient on all the approaches). For all the approaches there are two entry and two exit lanes. The lane width is 12ft (3.7m) per each lane. The inscribed diameter (D_i) is 48ft (14.65m). The posted speed limits on the approaches were 20mph (32km/hr). See Figure 6.9 for a detailed drawing of the roundabout. See Figure 6.10 for hourly turning movements.

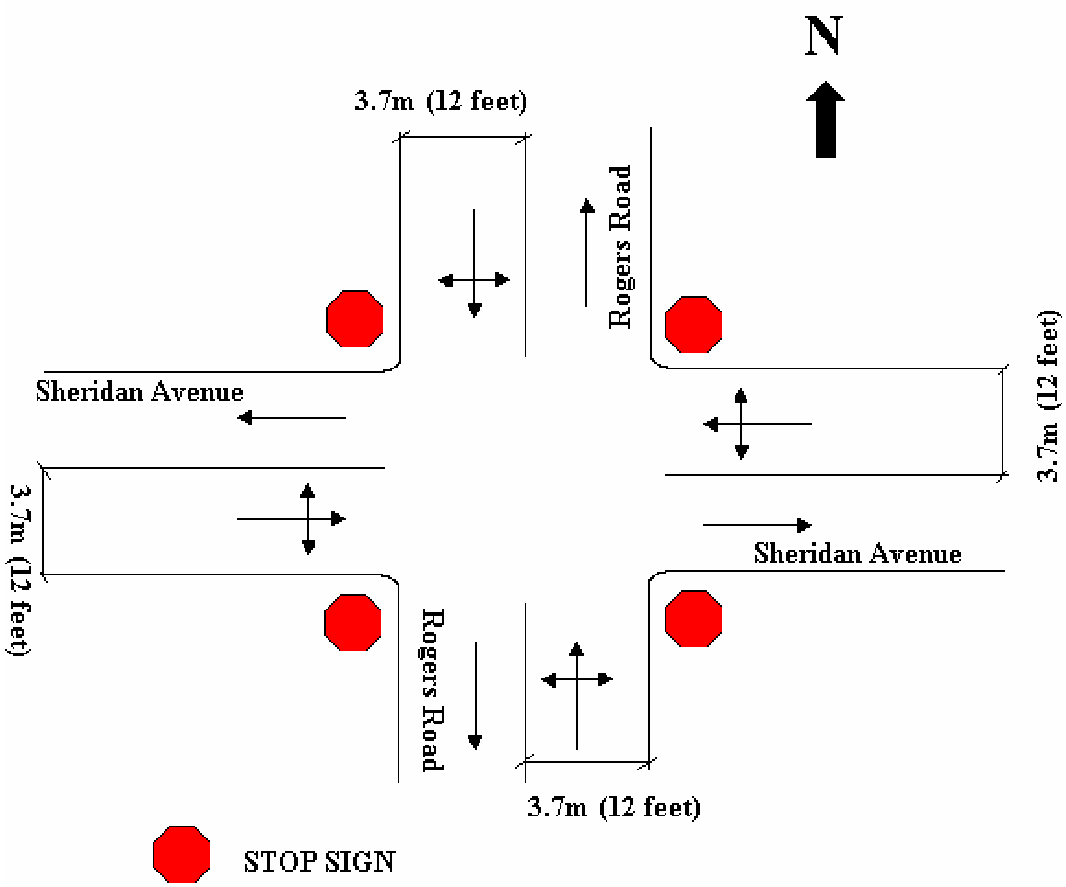


FIGURE 6.7: Rogers/Sheridan Intersection - Before Condition

AM Condition

North

```
                    43        65        3
                   <--'       |        '-->
        47    ---^            V                 ^---    10
West   148    --->                              <---   205    East
       144    ---v            ^                 v---    45
                   <--,       |        ,-->
                   193       126       30
```

South

PM Condition

North

```
                      80       126        10

                    <--'       |       '-->
        84    ---^             V                ^---     6
West   324    --->                              <---   249    East
       295    ---v             ^                v---    52
                    <--,       |       ,-->
                    192       102      49
```

South

FIGURE 6.8: Hourly Turning Movements for Rogers Road and Sheridan Avenue in Before Condition

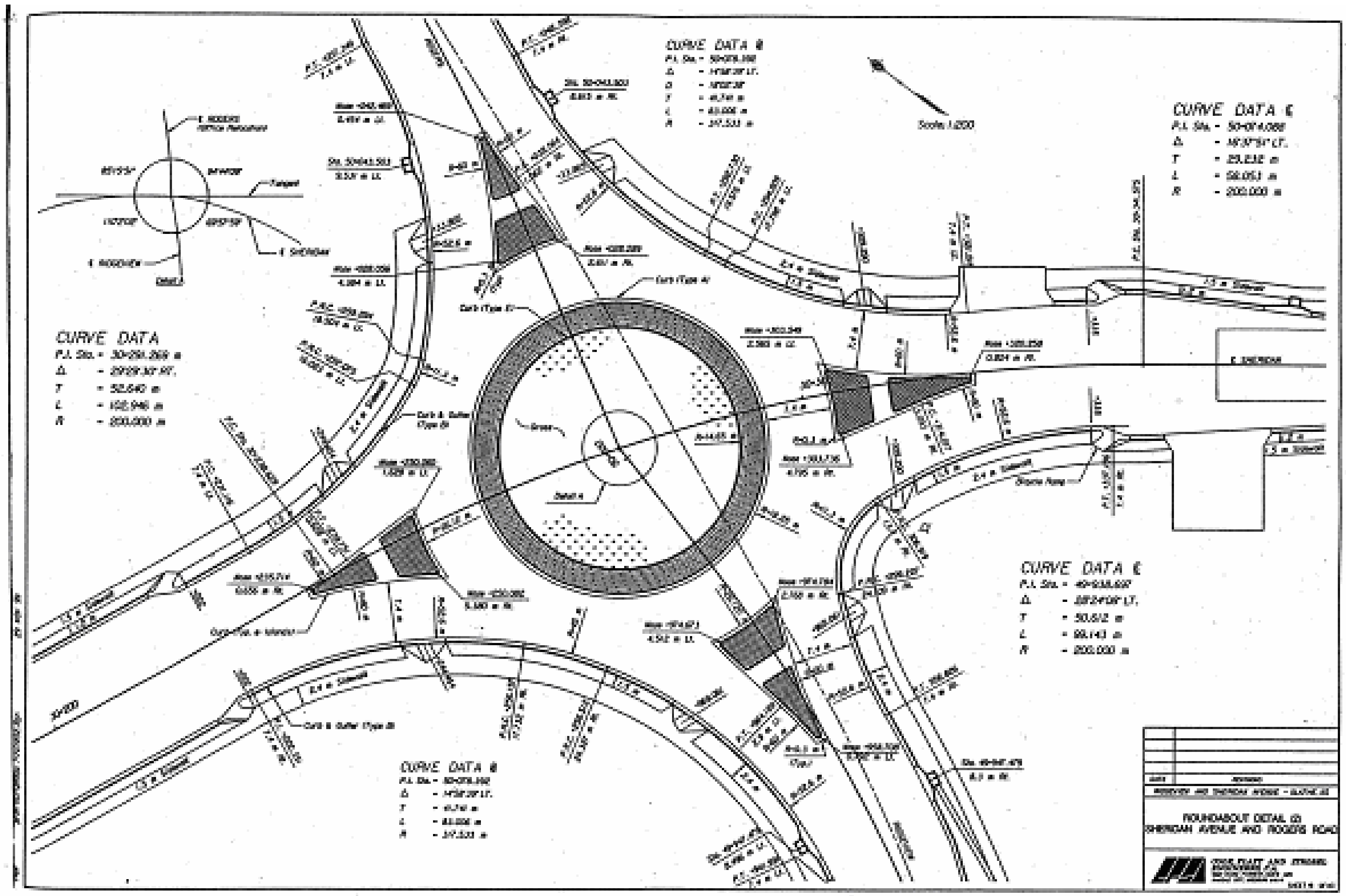


FIGURE 6.9: Rogers/Sheridan Intersection - After Condition
(Reduced from actual plan sheet provided by KDOT)

```
                    AM Condition
                       North

               42       81       6
              <--'       |      '-->
   39    ---^            V                ^---    13
West 163 --->                             <---   204  East
  195    ---v            ^                v---    53
              <--,       |      ,-->
              265       164      40

                       South
```

```
                    PM Condition
                       North
                  77      143      9

                <--       |      -->
   84    ---^             V               ^---    16
West 324 --->                             <---   250  East
  295    ---v             ^               v---    53
                <--       |      -->
                235      132     46

                       South
```

FIGURE 6.10: Hourly Turning Movements for Rogers Road and Sheridan Avenue in After Condition

- **Statistics for AM and PM periods- Rogers Road and Sheridan Avenue:** Tables 6.3 and 6.4 give the statistics for the observed AM and PM period hourly traffic volumes, respectively, with the intersection operating under before (AWSC) and after (RA) conditions and Figure 6.11 shows the traffic volume variation on a typical day with the intersection operating under each conditions.

TABLE 6.3: Descriptive Statistics for the AM Period Observed Hourly Traffic Volumes

Statistics	Traffic Volumes Veh/hr*	
	Intersection Treatments	
	AWSC	RA
Min	1220	1244
Mean	1569	1647
Max	1994	2024
Stdev	177	187
No.of Data points	46	34

*** Total Entering Vehicles**

TABLE 6.4: Descriptive statistics for the PM period observed hourly traffic volumes

Statistics	Traffic Volumes Veh/hr*	
	Intersection Treatments	
	AWSC	RA
Min	926	931
Mean	1174	1291
Max	1625	1738
Stdev	184	250
No.of Data points	28	26

*** Total Entering Vehicles**

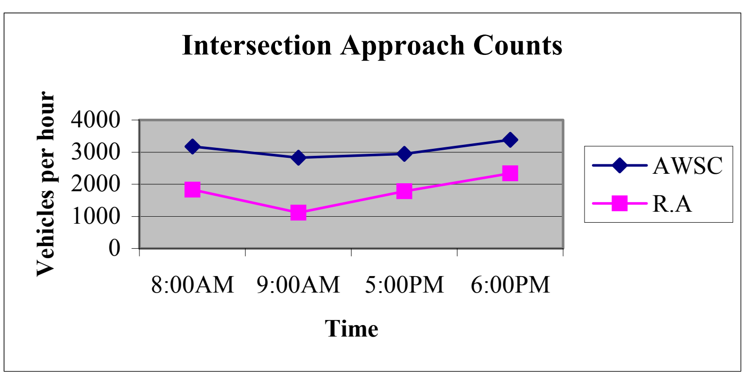


FIGURE 6.11: Traffic Volume Variation for a Typical Day

(Note: Lines in the figure are provided for reading convenience. No conclusions should be made that the lines indicate a statistical distribution or that there is a straight-line relationship between STIT time and vehicles per hour)

6.2.2 Lawrence

The intersection of Harvard Road and Monterey Way was studied. Harvard Road runs in the East-West direction while and ends at Monterey Way, which runs in the North-South direction. Figure 6.12 shows the location.

- **Hourly Volumes**: The traffic volume data was collected from 7:00AM to 9:00PM and from 4:00PM to 6:00PM on normal days for the before and after conditions. The before condition was with the intersection operating on normal days with AWSC. The after condition was with the intersection operating on normal days after the modern roundabout was in operation.

  The turning movement counts were obtained from video tapes for every 15-minute interval and recorded. Periods that had traffic less than 200 vehicles per hour were ignored in the analysis, as it was desired to study operational performance under higher volumes.

  The "before traffic volumes" were unusually low for reasons unknown to the research team. All efforts to prove the before and after data sets statistically similar failed. The assumptions of the F-test were not satisfied. Thus to proceed with the analysis, the before volumes were increased by 20% in the AM condition and 22% in the PM condition to make the sets statistically similar. The increase in volume was distributed among different turning movements in the same proportion as they were observed originally. The turning movement ratio was kept consistent with the original observation after the increase in volumes.
- **Before Condition:** Prior to the installation of the modern roundabout at this site the intersection was controlled by stop signs on all approaches (All Way Stop Control-AWSC).
- **Geometric parameters - Before condition:** In the before condition there was one approach lane and one exit lane in each of the approaches. The intersection is a three-legged intersection as shown in Figure 6.13. The lanes were 12ft (3.7m) wide. The terrain was flat with zero gradients on all the approaches. See Figure 6.14 for hourly turning movements.

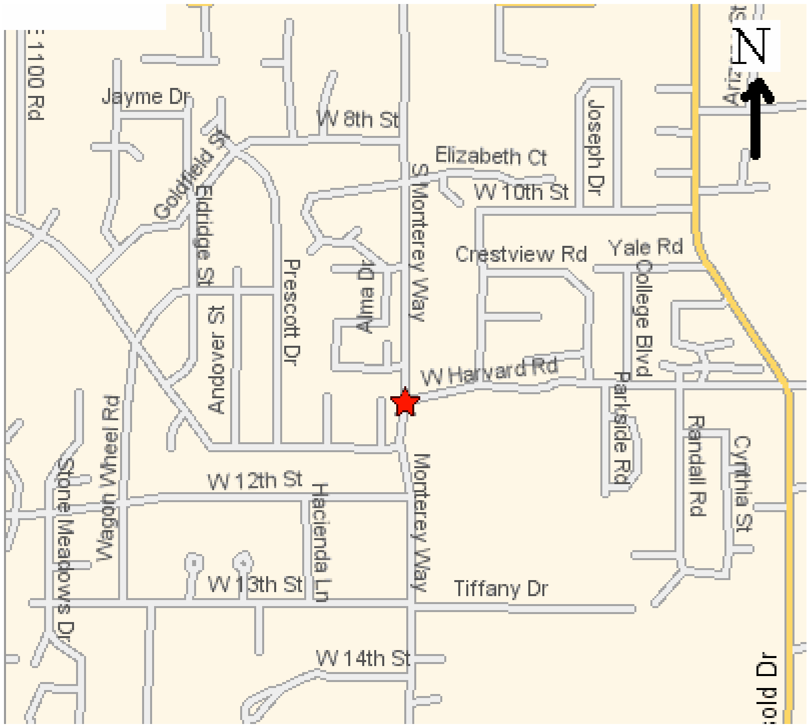


Figure 6.12: Figure Showing the Geographic Location of the Lawrence Roundabout
Source: Adopted from Maps.Yahoo.com

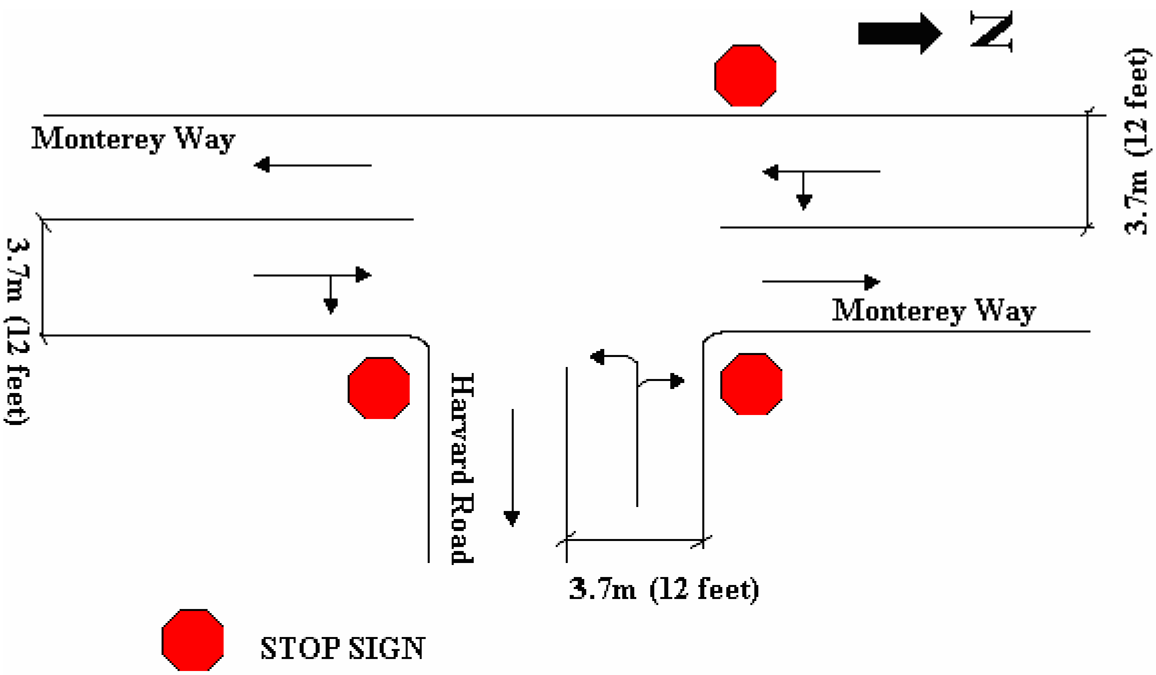


FIGURE 6.13: Harvard Road and Monterey Way in Before Condition

AM Condition

South	**91**	**--->**			**<---**	**92**	**North**
	55	**---v**			**v---**	**71**	
			<--	**-->**			
			45	**39**			

East

PM Condition

South	**138**	**--->**			**<---**	**134**	**North**
	75	**---v**			**v---**	**63**	
			<--	**-->**			
			82	**76**			

East

FIGURE 6.14: Hourly Turning Movements for Harvard Road and Monterey Way in Before Condition

- **After Condition:** In the after condition a modern roundabout was built. This roundabout is a single lane roundabout with a circular central island and three approaches. Key dimensions are given in the following section.
- **Geometric parameters - After condition:** The roundabout has a circulating lane width (W_c) of 12ft (3.7 m) on a flat terrain (zero gradient on all the approaches). For all the approaches there is one entry and one exit lanes. The lane width is 12ft (3.7 m). The inscribed diameter (D_i) is 45ft (15m). The posted speed limits on the approaches were 20mph (32km/hr).
- See Figure 6.15 for roundabout details. See Figure 6.16 for hourly turning movements.

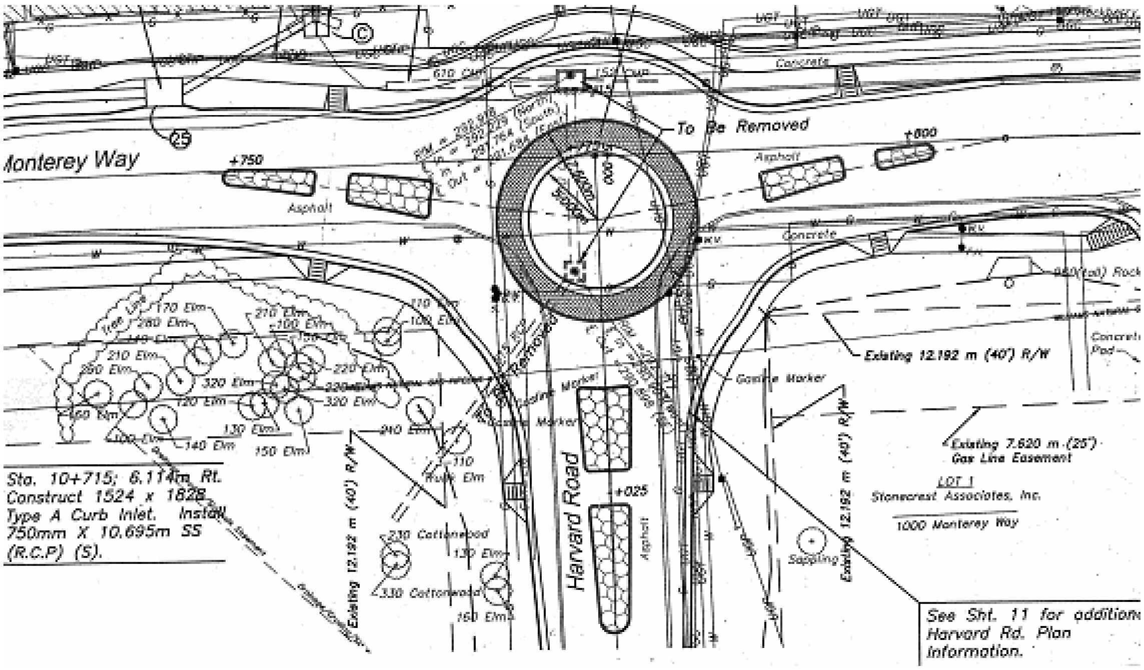


Figure 6.15: Harvard Road and Monterey Way Intersection - After Condition

(Reduced from actual plan sheet provided by City of Lawrence, KS)

AM Condition

South	**82**	**--->**			**<---**	**162**	**North**
	51	**---v**			**v---**	**34**	
			<--	**-->**			
			31	**36**			

East

PM Condition

South	**166**	**--->**			**<---**	**164**	**North**
	64	**---v**			**v---**	**44**	
			<--	**-->**			
			73	**57**			

East

Figure 6.16: Hourly Turning Movements for Harvard Road and Monterey Way in After Condition

- **Statistics for AM and PM periods - Harvard Road and Monterey Way:** Tables 6.5 and 6.6 give the statistics for the observed AM and PM period hourly traffic volumes, respectively, with the intersection operating under before (AWSC) and after (RA) conditions and Figure 6.17 shows the traffic volume variation on a typical day with the intersection operating under each conditions.

TABLE 6.5 Descriptive Statistics for the AM Period Observed Hourly Traffic Volumes

	*Traffic Volumes Veh/hr**	
Statistics	*Intersection Treatments*	
	AWSC	*RA*
Min	227	263
Mean	392	392
Max	636	447
Stdev	76	38
No. of Data points	50	44

*** Total Entering Vehicles**

TABLE 6.6: Descriptive Statistics for the PM Period Observed Hourly Traffic Volumes

	*Traffic Volumes Veh/hr**	
Statistics	*Intersection Treatments*	
	AWSC	*RA*
Min	412	442
Mean	568	567
Max	733	692
Stdev	80	85
No. of Data points	50	48

*** Total Entering Vehicles**

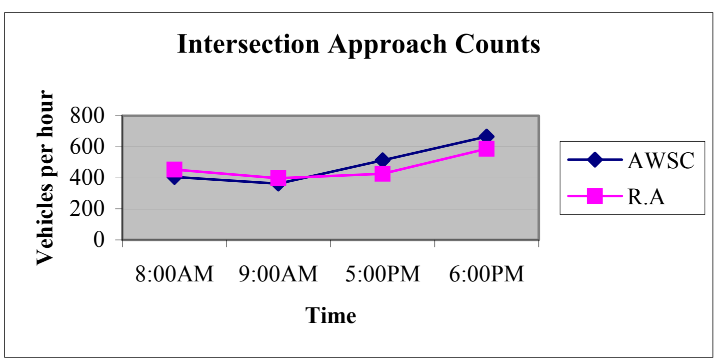


FIGURE 6.17: Traffic Volume Variation for a Typical Day

(Note: Lines in the figure are provided for reading convenience. No conclusions should be made that the lines indicate a statistical distribution or that there is a straight-line relationship between STIT time and vehicles per hour)

6.2.3 Paola

The site studied in Paola is the intersection of the Old K.C. Road, K-68 and Hedge Lane. The Old K.C. Road runs in the North-South direction. And the K-68 runs in the East-West direction. Hedge Lane runs in South-East-North-West direction, and in the before condition intersected K-68 just east of the K-68 and Old K.C. Road intersection. Hedge Road was relocated to intersect with K-68 at the roundabout. This roundabout location is different from the others as it has five legs, and is an intersection on the state highway. The location is shown in Figure 6.18.

FIGURE 6.18: Figure Showing the Geographic Location of the Paola Roundabout
Source: Maps.Yahoo.com

- **Hourly volumes**: The traffic volume data was collected from 7:00AM to 1:00PM and from 2:30PM to 8:30PM on normal days for the before and after conditions respectively. The before condition was with the intersection operating on normal days with AWSC. The after condition was with the intersection operating on normal days after the modern roundabout was in operation.

The turning movement counts were obtained from video tapes for every 15-minute interval and recorded. Periods that had traffic less than 200 vehicles per hour were ignored in the analysis, as it was desired to study the operational performance under higher volumes.

- **Before Condition:** Prior to the installation of the modern roundabout at this site the four-leg intersection was controlled by stop signs on all approaches (AWSC). All vehicles traversing through this intersection are required to stop before entering the intersection.
- **Geometric parameters - Before condition:** In the before condition there was one approach lane and one exit lane in each of the approaches. The lanes were 12ft (3.7 m) wide. The terrain was flat with zero gradients on all the approaches. See Figure 6.20 for hourly turning movements.

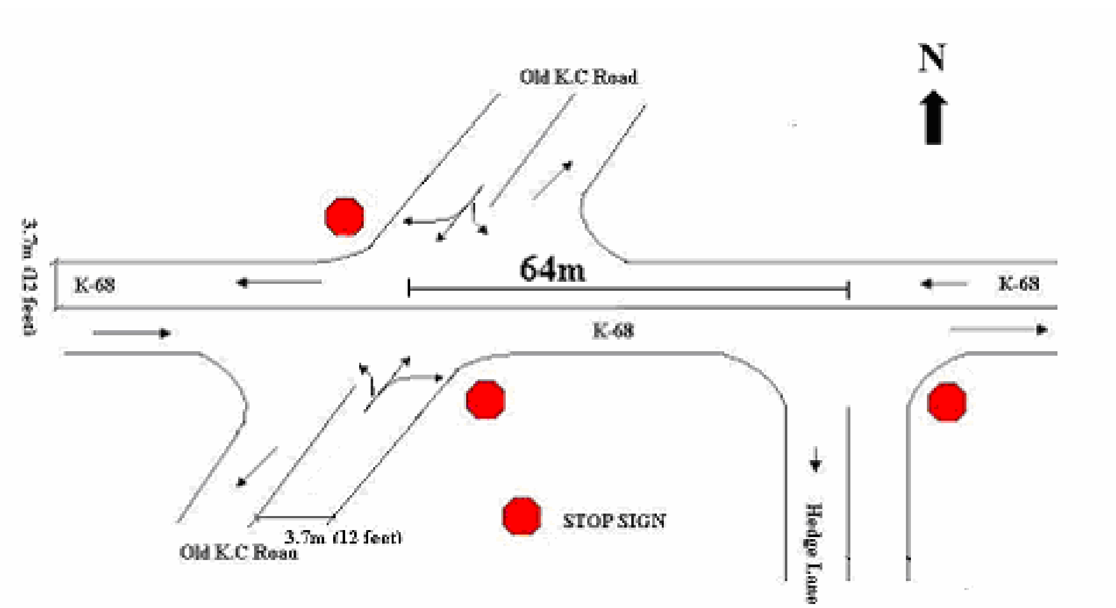


FIGURE 6.19: Old K.C. Road, K-68 and Hedge Lane - Before Condition

AM Condition

North

			1	**5**	**6**			
			<--'	**|**	**'-->**			
	1	**---^**		**V**		**^---**	**4**	
West	**77**	**--->**				**<---**	**56**	**East**
	32	**---v**		**^**		**v---**	**65**	
			<--	**|**	**-->**			
			25	**5**	**78**			

South

PM Condition

North

			1	**5**	**6**			
			<--'	**|**	**'-->**			
	1	**---^**		**V**		**^---**	**7**	
West	**70**	**--->**				**<---**	**86**	**East**
	34	**---v**		**^**		**v---**	**117**	
			<--	**|**	**-->**			
			37	**6**	**66**			

South

FIGURE 6.20: Hourly Turning Movements for Old K.C. Road and K-68 in Before Condition
(No counts were available for Hedge Lane)

- **After Condition:** In the after condition a modern roundabout was built. This roundabout in is a single lane roundabout with a circular central island. Hedge Road was realigned so that it would enter the roundabout. Therefore there are five approaches to this roundabout. Key dimensions are given in the following section.

- **Geometric parameters - After condition:** The roundabout has a circulating lane width (W_c) of 22ft (6.6 m) on a flat terrain (zero gradient on all the approaches). For all the approaches there is one entry and one exit lanes. The lane widths range from 15ft (4.6m) to 17ft (5.3m) for various approaches. The inscribed diameter (D_i) is 130ft (40m). The posted speed limits on the approaches were 20mph (32km/hr). See Figure 6.21 for roundabout details. See Figure 6.22 for hourly turning movements.

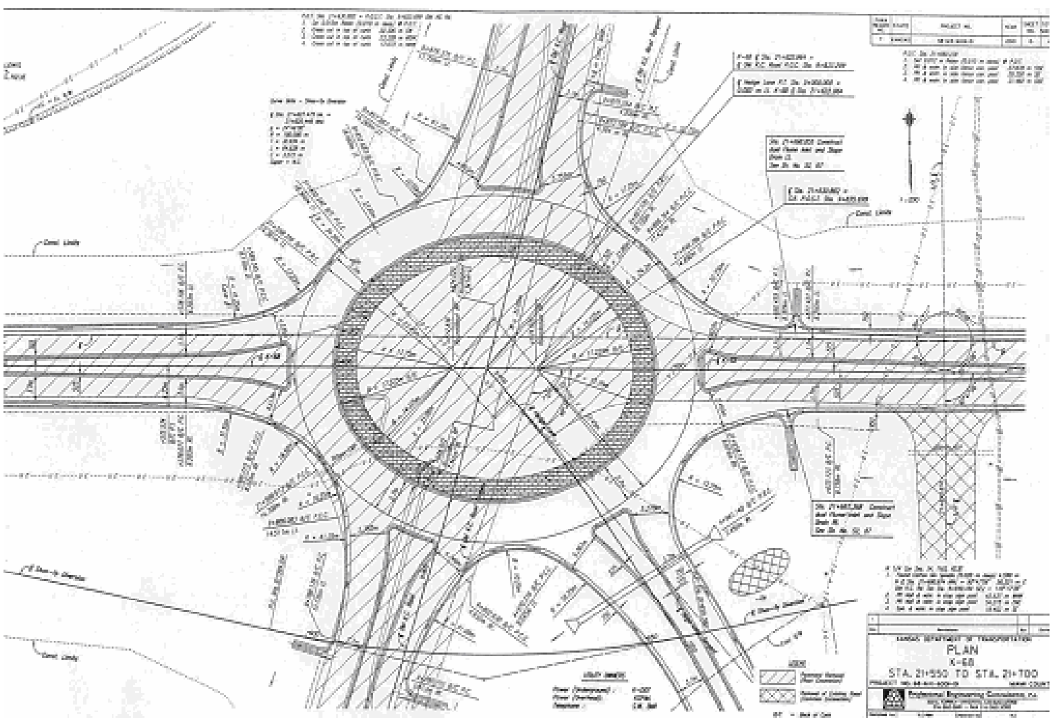
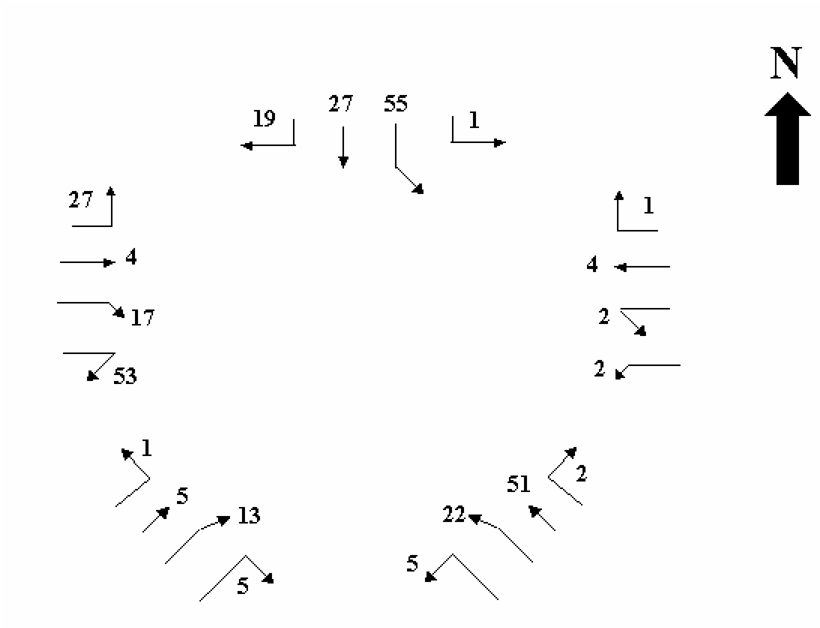


FIGURE 6.21: Old K.C. Road, K-68 and Hedge Lane Intersection - After Condition
(Reduced from the actual plans provided by KDOT)

AM Condition



PM Condition

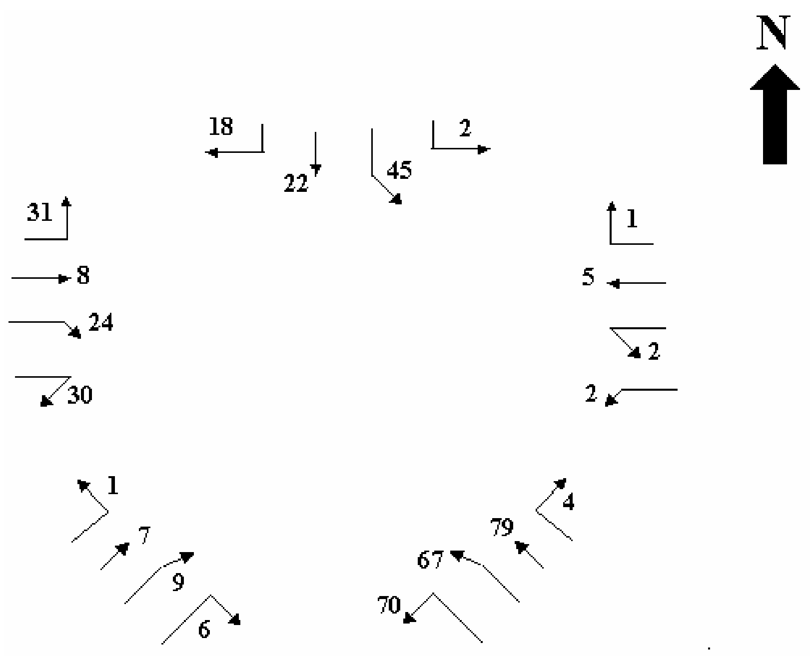


FIGURE 6.22: Hourly Turning Movements for Old K.C. Road and K-68 in After Condition

- **Statistics for AM and PM periods - Old K.C. Road and K-68:** Tables 6.7 and 6.8 give the statistics for the observed AM and PM period hourly traffic volumes, respectively, with the intersection operating under before (AWSC) and after (RA) conditions and Figure 6.23 shows the traffic volume variation on a typical day with the intersection operating under each conditions.

TABLE 6.7: Descriptive Statistics for the AM Period Observed Hourly Traffic Volumes

Statistics	*Traffic Volumes Veh/hr** *Intersection Treatments*	
	AWSC	***RA***
Min	271	271
Mean	370	365
Max	594	547
Stdev	101.24	78.84
No.of Data points	48	42

*** Total Entering Vehicles**

TABLE 6.8: Descriptive Statistics for the PM Period Observed Hourly Traffic Volumes

Statistics	*Traffic Volumes Veh/hr** *Intersection Treatments*	
	AWSC	***RA***
Min	192	93
Mean	427	418
Max	660	663
Stdev	138.82	153.74
No.of Data points	63	72

*** Total Entering Vehicles**

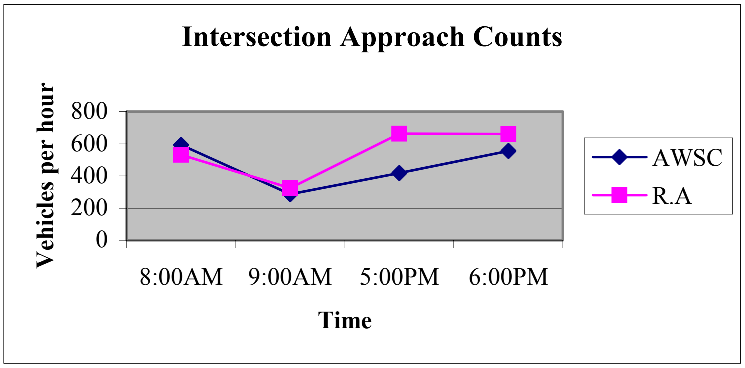


FIGURE 6.23: Traffic Volume Variation for a Typical Day

(Note: Lines in the figure are provided for reading convenience. No conclusions should be made that the lines indicate a statistical distribution or that there is a straight-line relationship between STIT time and vehicles per hour)

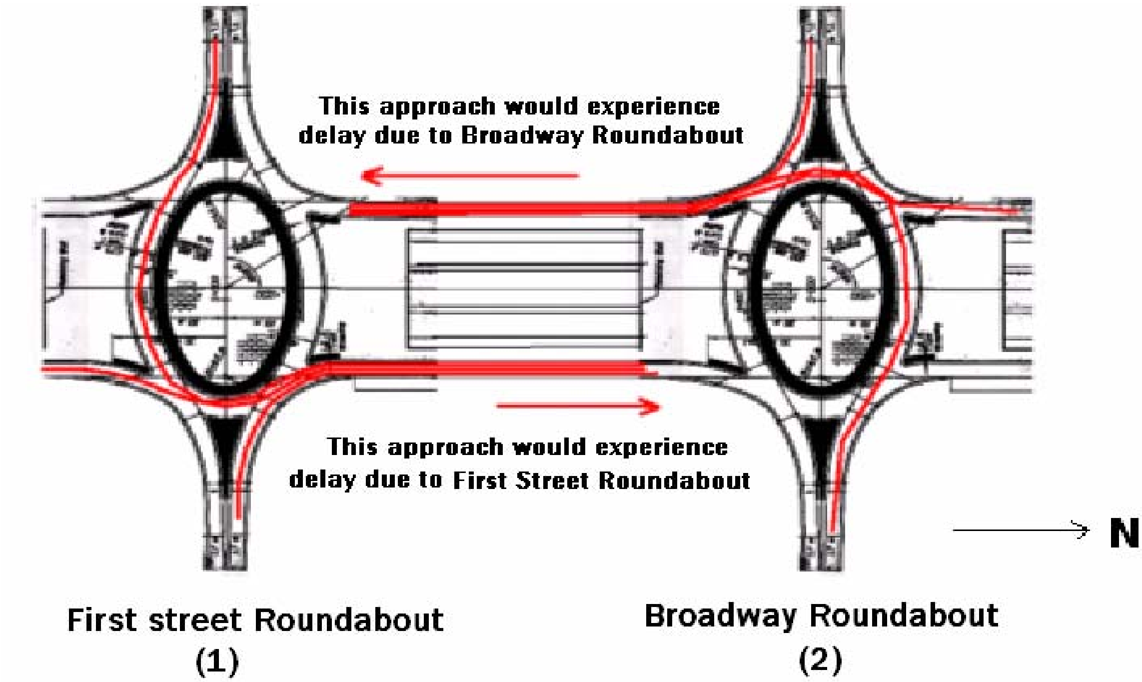
6.2.4 Newton

Two sites were studied in Newton, Kansas, the intersection of Interstate 135 (I-135) and Broadway and the intersection of Interstate 135 (I-135) and First Street.

- **Hourly Volumes**: The traffic volume data was collected from 7:00AM to 1:00PM and from 1:00PM to 7:00PM on normal week days for the before and after conditions.

  The turning movement counts were obtained for every 15-minute interval and recorded. Periods that had traffic less than 200 vehicles per hour were ignored in the analysis, as it was desired to study their operational performance under higher volumes. For the intersection of Broadway and Interstate (I-135) the before traffic volume was very low compared to the after traffic volumes for reasons unknown to the authors.

  The geometric configuration in the after condition was so different than the before condition that an actual before/after comparison would have been meaningless. So a theoretical comparison was made using the after traffic volumes to conditions that would have resulted if a traffic signal had been installed. (If a roundabout had not been constructed, traffic signals would have been installed). In the after condition the traffic coming from the south (northbound) that wants to exit at Broadway has to first pass through the roundabout at First Street and; likewise, the traffic coming from the north (southbound) that has to exit at First Street has to first pass through the Broadway roundabout. Hence the vehicles coming from the south approach into Broadway and vehicles coming from the north approach into First Street would experience delays due to both roundabouts. See Figure 6.24.



FIGURR 6.24: Figure Showing the Approaches that would Experience Delays Due to the Prior Roundabout

Also due to a lengthy construction period during which the roundabouts were constructed several months apart, the video-taping was done at different times at both these roundabouts. First the average delays are calculated for each roundabout independently.

When the average delay for the intersection was calculated, the delays for all approaches were averaged. Since one of the approaches experiences delays due to the roundabout at the prior intersection (i.e roundabout at Broadway for one at First Street and vice versa) the average delay of the prior intersection, affects the average intersection delay of the other intersection. However, in this study the delay due to the prior intersection was not taken into consideration. For example, the traffic approaching First Street is the sum of the Broadway south bound through traffic, right turning traffic from the Broadway west approach and the left turning traffic from the Broadway east approach. So each of these movements would experience delay at the Broadway roundabout. The amount of delay that these vehicles would experience is the average intersection delay of the Broadway roundabout. The delay was ignored when calculating the delay for the First street intersection.

The reason for ignoring the affect of prior intersection is because the taping was not done at the same time and as traffic volume and turning movement data are from two different time periods, and it was the opinion of the research team that combining them would give misleading results.

The same procedure was adopted for both Newton roundabouts and they are assumed to function independent of each other even though one of the approach movements goes through the previous roundabout.

See Figure 6.25 for geographic location of the roundabouts.

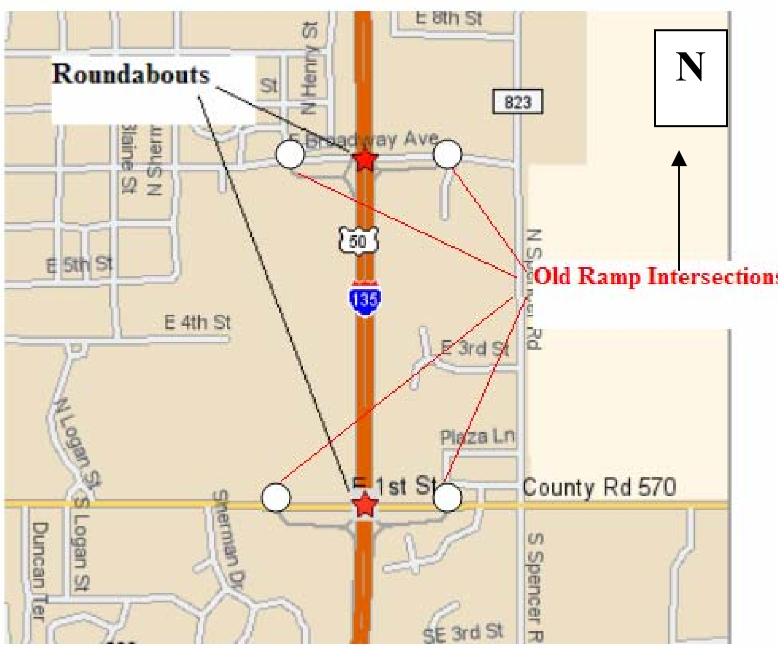


Figure 6.25: Figure Showing the Geographic Locations of the Two Newton, Kansas Roundabouts and Old Ramp Intersection

Source: Adopted from Maps.Yahoo.com

1. **<u>NEWTON: Location A</u>:** Intersection of Interstate 135 (I-135) and Broadway

- **Before Condition:** In the before condition, Broadway crossed over interstate (I-135). There were standard entrance and exit ramps away from interstate (I-135). The before condition was theoretically analyzed assuming that if the roundabouts had not been built, a traffic signal would have been provided. The after condition volumes were used in analysis. This was done because the before volumes were so much different (lower) and the before and after flows were not directly comparable.
- **Geometric parameters - Before condition:** In the before condition there was one approach lane and one exit lane in each of the approaches. The lanes were 12ft (3.7m) wide. The terrain was flat with zero gradients on all the approaches. See figure 6.26 for hourly turning movements.
- **After Condition:** In the after condition a modern roundabout was built, replacing standard entrance and exit ramps. The interstate now crosses over Broadway. The exit for both Broadway and First street for northbound interstate traffic, is south of First Street. The northbound entrance ramp to the highway from First Street and Broadway is north of Broadway. For southbound interstate traffic, the exit for Broadway and First street, is north of Broadway. The southbound entrance ramp to the highway from Broadway and First street is south of First Street. Connector roads run on the east and west sides of the highway, linking the Broadway and First street roundabouts. The roundabouts are single lane roundabout with an oval central island. See Figure 6.27. Key dimensions are given in the following section.
- **Geometric parameters - After condition:** The roundabout has a circulating lane width (W_c) of 18ft (5.5m) on a flat terrain (zero gradient on all the approaches). There is one circulating lane. For all the approaches there is one entry and one adjacent exit lane. The lane width is 12ft (3.7m) for Broadway entrance and exit ramps, exit of East Connector road, and West connector road. It is 14ft (4.3m) for East connector road and exit of West Connector road. See Figure 6.27 for detailed drawing of the roundabout. See Figure 6.28 for hourly turning movements.

AM Condition

```
                          North

                   3        4        3
                 <--'       |       '-->
       3    ---^            V              ^---     5
West   91   --->                           <---     118   East
       55   ---v            ^              v---     21
                 <--,       |       ,-->
                   13       1        21

                          South
```

PM Condition

```
                          North

                     2      5      3
                   <--'     |     '-->
       160  ---^            V              ^---     7
West   4    --->                           <---     196   East
       93   ---v            ^              v---     22
                   <--,     |     ,-->
                     18     2      20

                          South
```

FIGURE 6.26: Original Hourly Turning Movements for I-135 and Broadway in Before Condition

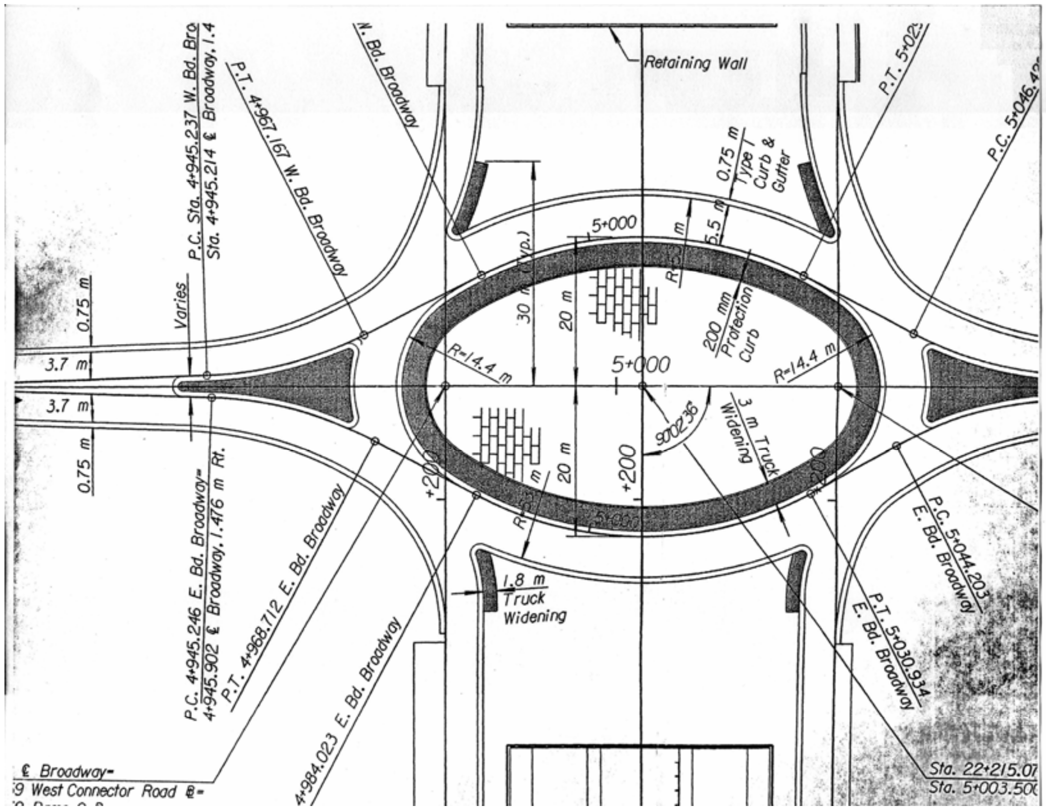


FIGURE 6.27: Interstate (I-135) and Broadway Intersection - After Condition

(Reduced from the actual plan)

AM Condition

North

```
                         12      16      12
                        <--'      |      '-->
       12      ---^               V              ^---     20
West   364     --->                              <---     472    East
       220     ---v               ^              v---     84
                        <--,      |      ,-->
                         52       4      84
```

South

PM Condition

North

```
                          8      20      12
                        <--'      |      '-->
       16      ---^               V              ^---     28
West   372     --->                              <---     784    East
       256     ---v               ^              v---     88
                        <--,      |      ,-->
                         72       8      80
```

South

FIGURE 6.28: Turning Movements for I-135 and Broadway in After Condition

- **Statistics for AM and PM periods: I-135 and Broadway:** Tables 6.9 and 6.10 give the statistics for the observed AM and PM period hourly traffic volumes, respectively, with the intersection operating under before (Signal- Assumed) and after (RA) conditions. As previously explained, the after volumes were used for the before condition theoretical analysis. Hence the values shown for before and after conditions are exactly the same. Figure 6.29 shows the traffic volume variation on a typical day.

TABLE 6.9 Descriptive Statistics for the AM Period Observed Hourly Traffic Volumes

	*Traffic Volumes Veh/hr**	
Statistics	***Intersection Treatments***	
	Signal	***RA***
Min	1392	1392
Mean	1541	1541
Max	1556	1556
Stdev	74	74
No. of Data points	9	9

*** Total Entering Vehicles**

TABLE 6.10: Descriptive Statistics for the PM Period Observed Hourly Traffic Volumes

	*Traffic Volumes Veh/hr**	
Statistics	***Intersection Treatments***	
	Signal	***RA***
Min	1593	1593
Mean	1857	1857
Max	1971	1971
Stdev	185	185
No.of Data points	10	10

*** Total Entering Vehicles**

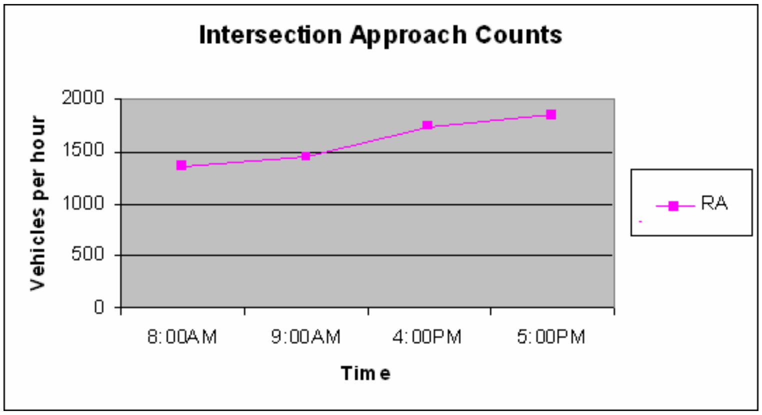


FIGURE 6.29: Traffic Volume Variation for a Typical Day

(Note: Lines in the figure are provided for reading convenience. No conclusions should be made that the lines indicate a statistical distribution or that there is a straight-line relationship between STIS time and vehicles per hour)

2. **<u>NEWTON: Location B</u>:** Intersection of Interstate 135 (I-135) and First Street

- **Before Condition:** In the before condition, the bridge at First Street crossed over the interstate (I-135). There were standard entrance and exit ramps away from the interstate (I-135). As in the case of Broadway (discussed previously), the before condition was theoretically analyzed assuming that if the roundabouts had not been built, a traffic signal would have been provided. The after condition volumes were used in analysis as explained previously for the Broadway roundabout before condition.
- **Geometric parameters - Before condition:** In the before condition there was one approach lane and one exit lane in each of the approaches. The lanes were 12ft (3.7m) wide. The terrain was flat with zero gradients on all the approaches.
- **After Condition:** In the after condition a modern roundabout was built, replacing standard entrance and exit ramps. The interstate now crosses over First Street. The exit for both Broadway and First street, for northbound interstate traffic, is south of First Street. The northbound entrance ramp to the highway from First street and Broadway is north of Broadway. For southbound interstate traffic, the exit for Broadway and First street, is north of

Broadway. The southbound entrance ramp to the highway from Broadway and First street is south of First Street. Connector roads run on the east and west sides of the highway, linking the Broadway and First street roundabouts. The roundabouts are single lane roundabout with an oval central island. Key dimensions are given in the following section.

- **Geometric parameters - After condition:** The roundabout has a circulating lane width (W_c) of 18ft (5.5m) on a flat terrain (zero gradient on all the approaches). There is one circulating lane. For all the approaches there is one entry and one adjacent exit lane. The lane width is 12ft (3.7m) for First Street entrance and exit ramps. It is 14ft (4.3m) for East connector road and West Connector roads. See Figure 6.30 for detailed drawing of the roundabout. See Figure 6.31 for hourly turning movements.

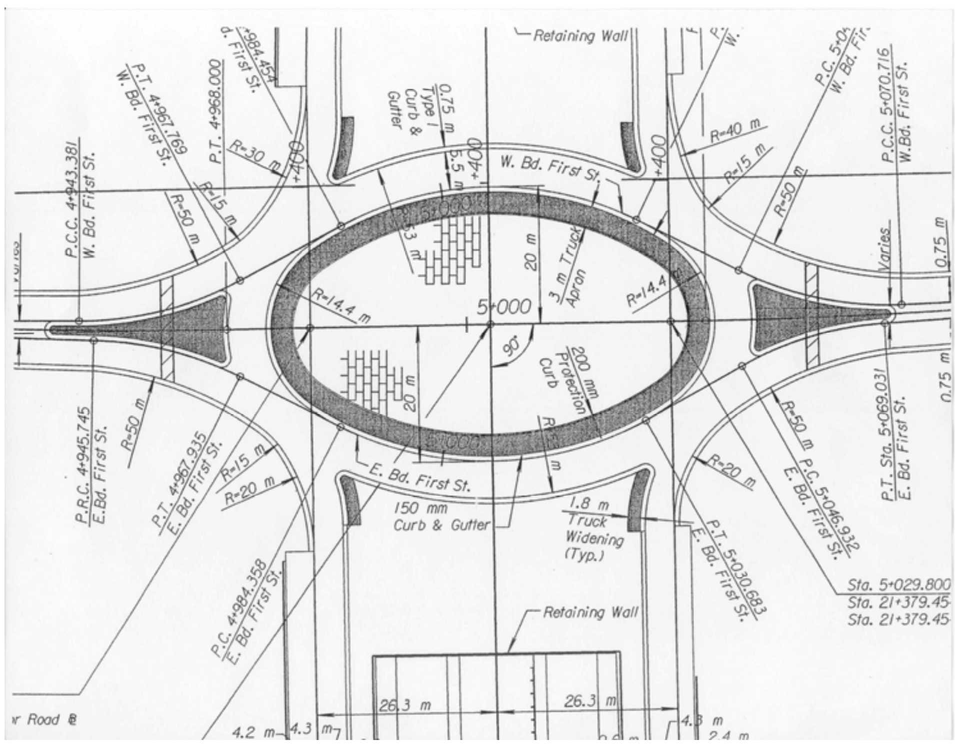


Figure 6.30: Interstate (I-135) and First Street Intersection - After Condition

(Reduced from the actual plan)

AM Condition
North

```
                    12        16        12
                   <--'       |        '-->
     12     ---^              V               ^---     20
West 364    --->                              <---     472   East
     220    ---v              ^               v---     84
                   <--,       |        ,-->
                    52        4         84
```

South

PM Condition
North

```
                     8        20       12
                    <--'      |       '-->
     16     ---^              V               ^---     28
West 372    --->                              <---     784   East
     256    ---v              ^               v---     88
                    <--,      |       ,-->
                     72       8        80
```

South

FIGURE 6.31: Turning Movements for I-135 and First Street in After Condition

- **Statistics for AM and PM periods: I-135 and First Street:** Tables 6.11 and 6.12 give the statistics for the observed AM and PM period hourly traffic volumes, respectively, with the intersection operating under before (Signal-Assumed) and after (RA) conditions. As previously explained, the after volumes were used for the before condition, theoretical analysis. Hence the values shown for before and after conditions are exactly the same. Figure 6.32 shows the traffic volume variation on a typical day with the intersection operating under after condition.

TABLE 6.11 Descriptive Statistics for the AM Period Observed Hourly Traffic Volumes

	Traffic Volumes Veh/hr*	
Statistics	***Intersection Treatments***	
	Signal	***RA***
Min	575	575
Mean	744	744
Max	901	901
Stdev	142	142
No. of Data points	8	8

*** Total Entering Vehicles**

TABLE 6.12 Descriptive Statistics for the PM Period Observed Hourly Traffic Volumes

	Traffic Volumes Veh/hr*	
Statistics	***Intersection Treatments***	
	Signal	***RA***
Min	846	846
Mean	962	962
Max	1079	1079
Stdev	84	84
No.of Data points	10	10

*** Total Entering Vehicles**

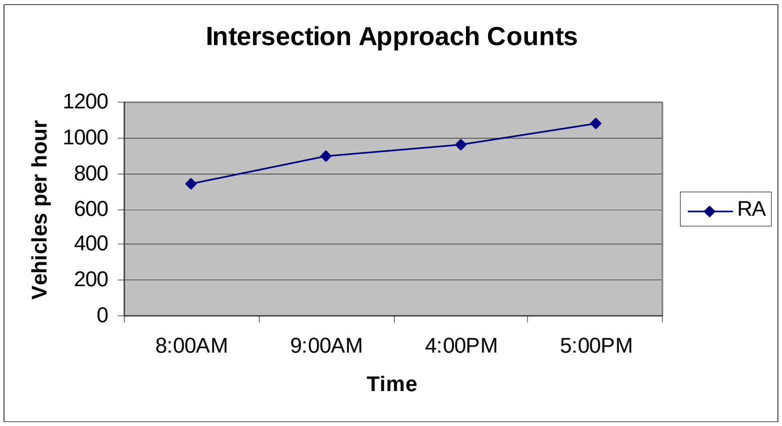


FIGURE 6.32: Traffic Volume Variation for a Typical Day

(Note: Lines in the figure are provided for reading convenience. No conclusions should be made that the lines indicate a statistical distribution or that there is a straight-line relationship between STIS time and vehicles per hour)

See Figures 6.33 for a sketch of the proposed grade changes with a roundabout and Figure 6.34 for an artistic sketch of the assumed signal alternative used, for the before condition at the two Newton intersections.

FIGURE 6.33: Proposed Intersection Alternative 1: Roundabout
(Source: KDOT)

FIGURE 6.34: Proposed Intersection Alternative 2: Traffic Signal
(Artist Sketch, source KDOT)

6.2.5 Topeka

Two sites were studied in Topeka, Kansas, the intersection of Rice Road and Interstate 70 (I-70) and the intersection of US-75 and NW 46th Street.

1. TOPEKA: LOCATION 1: Intersection of Rice Road and Interstate 70 (I-70)

The site on Rice Road, in Topeka, Kansas, has two roundabouts. One of them is north of Interstate 70 (I-70) and one south of I-70. Rice Road runs in a North-South direction and has ramps for entering and exiting I-70. The south roundabout serves on and off ramps to and from eastbound I-70 traffic exiting to Rice Road. The north Roundabout serves on and off ramps to and from westbound I-70.

- **Hourly Volumes**: The traffic volume data was collected from 7:00AM to 12:00 noon and from 3:00PM to 9:00PM on normal week days for the before and after conditions.

  The turning movement counts were obtained for every 15-minute interval and recorded. Periods that had traffic less than 200 vehicles per hour were ignored in the analysis, as it was desired to study the roundabouts operational performance under higher volumes. Visual analysis of the roundabouts confirmed that the heavy truck traffic from a nearby truck terminal had no problems and traversed the roundabouts easily and efficiently.

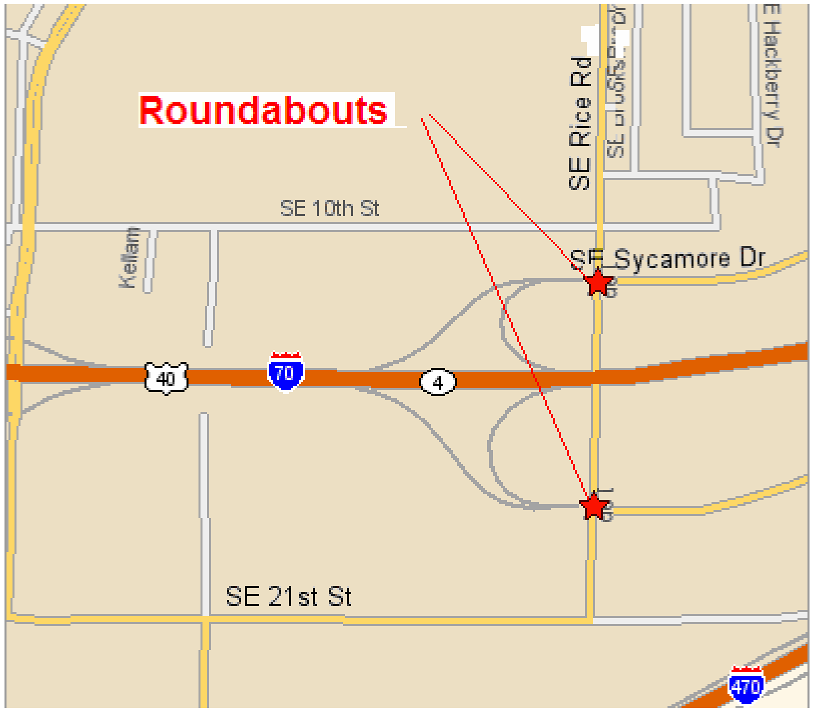


FIGURE 6.35: Geographic Locations of the Two Rice Road Roundabouts
in Topeka, Kansas
Source: Adopted from Maps.Yahoo.com

- **Before Condition:** These are new intersections. As a part of the alignment of I-70, an interchange was planned at Rice Road. It was decided to design roundabouts for traffic control at the intersection of Rice road and the on/off ramps. The before condition was theoretically analyzed assuming that if the roundabouts had not been built, Two -Way Stop Control (TWSC) would have been provided.
- **Geometric parameters - Before condition:** In the before condition, for the North Roundabout and South Roundabouts the approach lanes were 3.7m (12 feet) wide. The terrain was flat with zero gradients on all the approaches. See figure 6.36 and 6.37 for hourly turning movements.

AM Condition

North

```
                        22        17        0
                       <--'       |        '-->
      23       ---^               V                  ^---      25
West   0       --->                                  <---     193    East
      20       ---v               ^                  v---      21
                       <--,       |        ,-->
                        12        12        0
```

South

PM Condition

North

```
                        25        29        0

                       <--'       |        '-->
       9       ---^               V                  ^---       8
West   0       --->                                  <---      74    East
      39       ---v               ^                  v---      14
                       <--,       |        ,-->
                        13        17        0
```

South

FIGURE 6.36: Hourly Turning Movements for North Rice Road in Before Condition with assumed TWSC as the Traffic Control

AM Condition

```
                         North
                    25    52    10

                   <--'    |    '-->
      39   ---^            V           ^---   0
West  47   --->                        <---   0        East
       7   ---v            ^           v---   0
                   <--,    |    ,-->
                    22    34     2

                         South
```

PM Condition

```
                         North

                    38    50    44
                   <--'    |    '-->
      55   ---^            V           ^---      0
West 308   --->                        <---      0    East
      15   ---v            ^           v---      0
                   <--,    |    ,-->
                    12    24    17

                         South
```

FIGURE 6.37: Hourly Turning Movements for South Rice Road in Before Condition with Assumed TWSC as the Traffic Control

- **After Condition:** For the after condition, the modern roundabout that was constructed was analyzed. Both the north and south roundabouts are two-lane, one lane roundabouts with a circular central island. The circulating section of the roundabout has two-lanes for some portion and one-lane for the remaining portion. Key dimensions are given in the following section. See figures 6.41 and 6.42 for hourly turning movements.
- **Geometric parameters - North Roundabout:** The roundabout has a circulating lane of varying width. The portion with only one circulating lane has (W_c) of 15ft (4.57m) and for the portion with two circulating lanes, each lane with a width of (W_c) of 15ft (4.57m), on a flat terrain (zero gradient on all the approaches). For the North approach there is one entry and two adjacent exit lanes. For the South Approach there is one entry and one adjacent exit lanes. For the East approach there are two approach lanes and no adjacent exit lanes. For the West approach there are two approach lanes and one adjacent exit lane. The lane width is 12ft (3.7m). See Figure 6.39 for detailed drawing of the roundabout.
- **Geometric parameters - South Roundabout:** The roundabout has a circulating lane of varying width. The portion with only one circulating lane has (W_c) of 15ft (4.57m) and the portion with two circulating lanes has each lane with a width of (W_c) of 15ft (4.57m), on a flat terrain (zero gradient on all the approaches). For the North and South approaches there are two entry and one adjacent exit lane. For the East approach there are no approach lanes and two adjacent exit lanes. For the West approach there are two approach lanes and two adjacent exit lanes. The lane width is 12ft (3.7m). See Figures 6.40 for detailed drawing of the roundabout.

Figure 6.38 shows the aerial view of the roundabouts.

FIGURE 6.38: Aerial View of the Realigned Interstate 70 (I-70) and Rice Road Intersection Roundabouts
(Source KDOT)

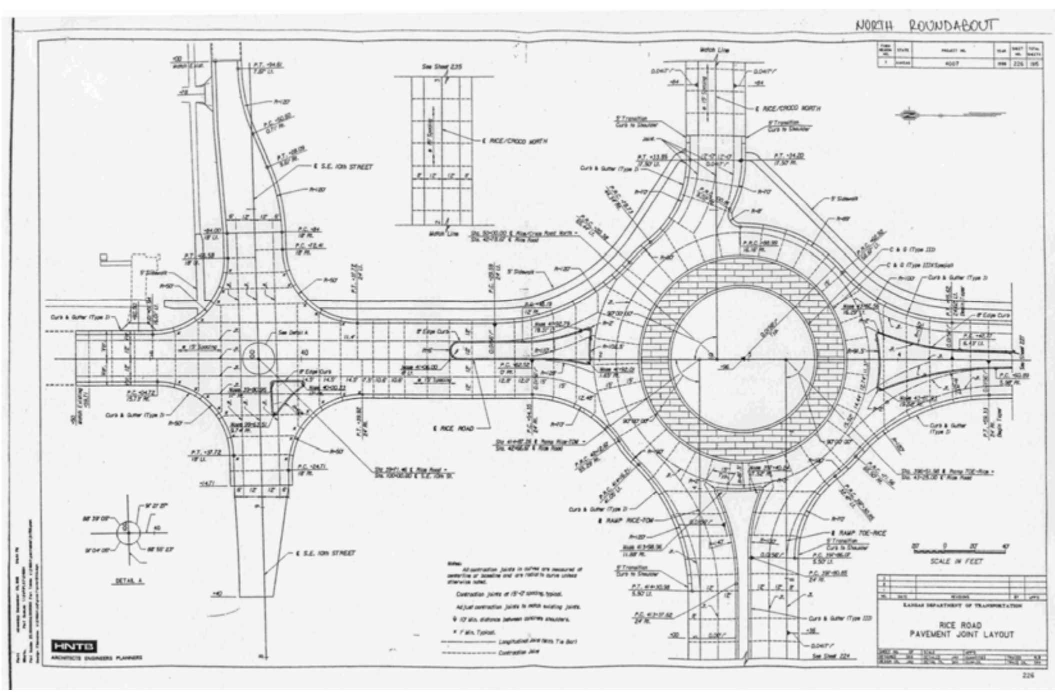


FIGURE 6.39: Interstate 70 (I-70) and Rice Road Intersection North Roundabout - After Condition
(Reduced from the actual plan provided by KDOT)

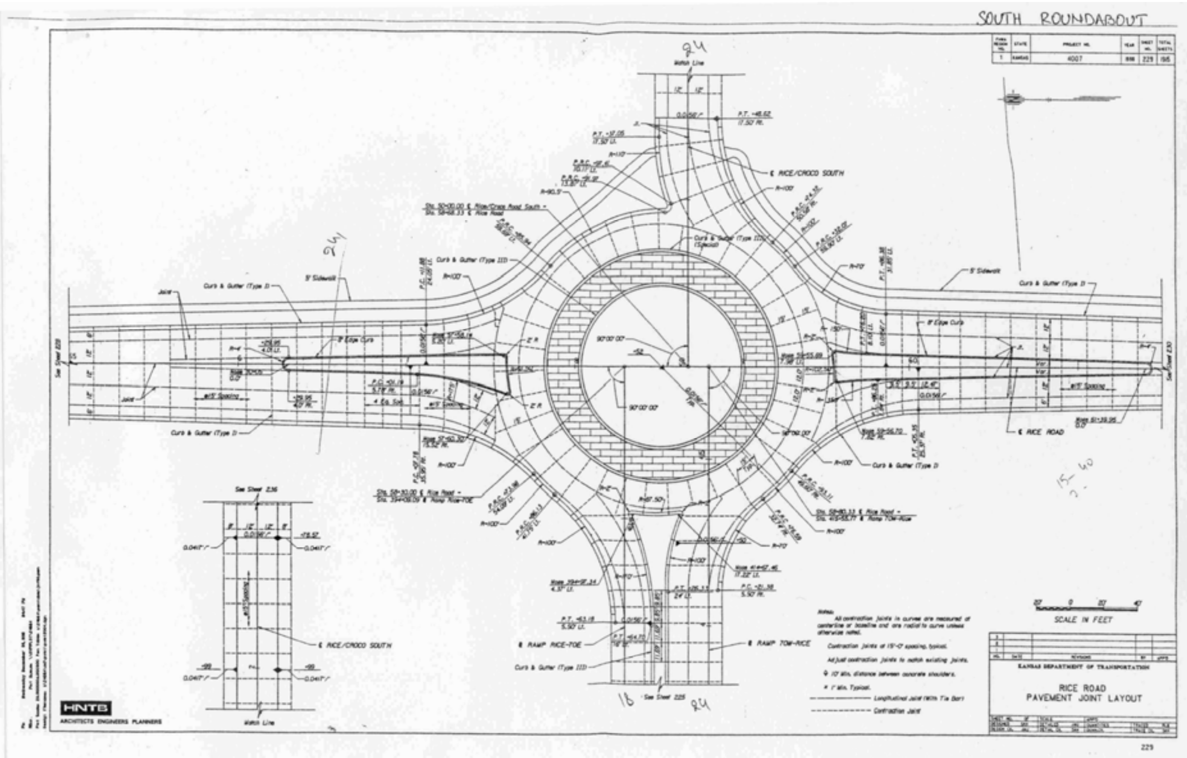


FIGURE 6.40: Interstate 70 (I-70) and Rice Road Intersection South Roundabout - After Condition

(Reduced from the actual plan provided by KDOT)

AM Condition

```
                         North

                  17       9       0
                 <--'      |      '-->
      20    ---^           V             ^---    5
West  0     --->                         <---    86    East
      3     ---v           ^             v---    12
                 <--,      |      ,-->
                  14      13       0

                         South
```

PM Condition

```
                         North
                  26      32       0

                 <--'      |      '-->
      11    ---^           V             ^---    8
West  0     --->                         <---    87    East
      35    ---v           ^             v---    23
                 <--,      |      ,-->
                  22      24       0

                         South
```

FIGURE 6.41: Hourly Turning Movements for North Rice Road in After Condition with the Roundabout as Designed

AM Condition

North

```
                      26        26        18
                     <--'        |        '-->
      27    ---^                 V                  ^---     0
West  96    --->                                    <---     0    East
      14    ---v                 ^                  v---     0
                     <--,        |        ,-->
                      7         22        12
```

South

PM Condition

North

```
                      80        65        41

                     <--'        |        '-->
      56    ---^                 V                  ^---     0
West  357   --->                                    <---     0    East
      14    ---v                 ^                  v---     0
                     <--,        |        ,-->
                      14        38        15
```

South

FIGURE 6.42: Hourly Turning Movements for South Rice Road in After Condition with the Roundabout as Designed

- **Statistics for AM and PM periods: North Roundabout:** Tables 6.13 and 6.14 give the statistics for the observed AM and PM period hourly traffic volumes, respectively, with the intersection operating under the theoretically assumed (TWSC) and actual after (RA) conditions. Since the before condition was a theoretical with no volume counts, so the after volumes were used for the analysis. Hence the values shown for before and after conditions are the same.

Figure 6.43 shows the traffic volume variation on a typical day with the intersection operating under after condition.

TABLE 6.13: Descriptive Statistics for the AM Period Observed Hourly Traffic Volumes

Statistics	*Traffic Volumes Veh/hr** *Intersection Treatments*	
	TWSC	***RA***
Min	616	616
Mean	643	643
Max	669	669
Stdev	23	23
No. of Data points	20	20

*** Total Entering Vehicles**

TABLE 6.14: Descriptive Statistics for the PM Period Observed Hourly Traffic Volumes

Statistics	*Traffic Volumes Veh/hr** *Intersection Treatments*	
	TWSC	***RA***
Min	255	255
Mean	292	292
Max	320	320
Stdev	31	31
No.of Data points	28	28

*** Total Entering Vehicles**

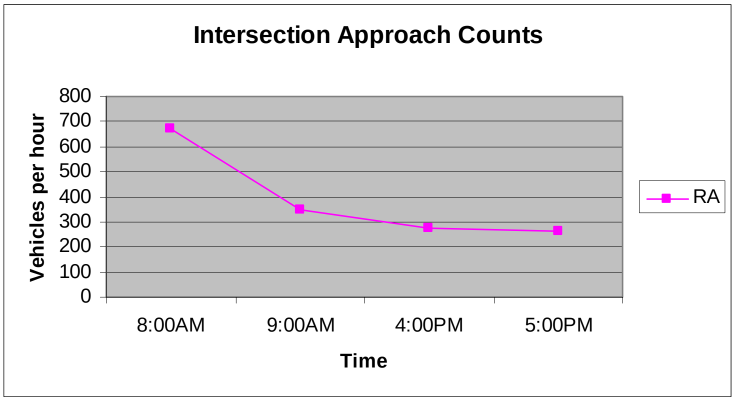


FIGURE 6.43: Traffic Volume Variation for a Typical Day
(Note: Lines in the figure are provided for reading convenience. No conclusions should be made that the lines indicate a statistical distribution or that there is a straight-line relationship between STIS time and vehicles per hour)

- **Statistics for AM and PM periods: South Roundabout:** Tables 6.15 and 6.16 give the statistics for the observed AM and PM period hourly traffic volumes, respectively, with the intersection operating under theoretically assumed (TWSC) and actual after (RA) conditions. Since the before condition was a theoretical with no volume counts, so the after volumes were used for the analysis. Hence the values shown for before and after conditions are the same. Figure 6.44 shows the traffic volume variation on a typical day with the south intersection operating under the after condition.

TABLE 6.15 Descriptive Statistics for the AM Period Observed Hourly Traffic Volumes

	Traffic Volumes Veh/hr*	
Statistics	***Intersection Treatments***	
	TWSC	***RA***
Min	238	238
Mean	262	262
Max	291	291
Stdev	20	20
No. of Data points	30	30

*** Total Entering Vehicles**

TABLE 6.16: Descriptive Statistics for the PM Period Observed Hourly Traffic Volumes

	Traffic Volumes Veh/hr*	
Statistics	***Intersection Treatments***	
	TWSC	***RA***
Min	542	542
Mean	628	628
Max	704	704
Stdev	61	61
No.of Data points	48	48

*** Total Entering Vehicles**

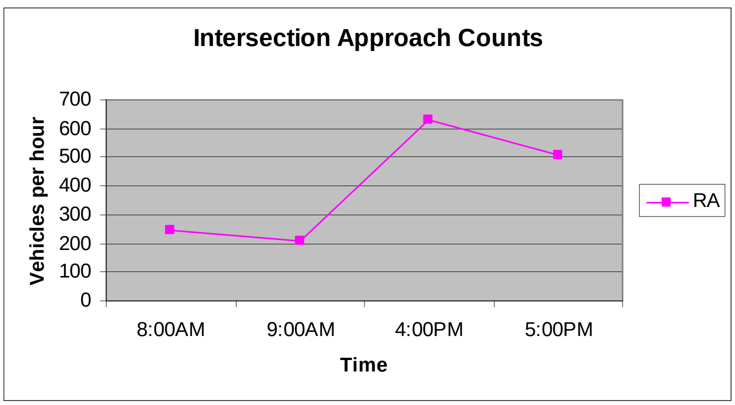


FIGURE 6.44: Traffic Volume Variation for a Typical Day
(Note: Lines in the figure are provided for reading convenience. No conclusions should be made that the lines indicate a statistical distribution or that there is a straight-line relationship between STIS time and vehicles per hour)

2. TOPEKA: Location 2: The site studied is the intersection of US-75 and NW 46th Street. US-75 runs in the North-South direction and NW 46th Street runs in the East-West direction. The location is shown in Figure 6.45.

- **Hourly volumes:** The traffic volume data was collected from 7:00AM to 1:00PM and from 1:00PM to 7:00PM on normal week days for the before and after conditions.

The turning movement counts were obtained for every 15-minute interval and recorded. In the after condition, it was not possible to extract the left turning movements from all directions due to an improper view from the cameras, i.e., it had not been placed and/or aimed for optimum viewing. For each approach the through and left turning volumes were counted as through and then it was assumed that the left turning percentage was the same as the before case and, the left turn percentages from the before volumes were used and applied to the through after volumes . After the left turning volumes were calculated, the through volumes were obtained by subtracting the left turn volumes from the earlier volumes which had the combined through and left volumes. After these adjustments, the analysis was performed.

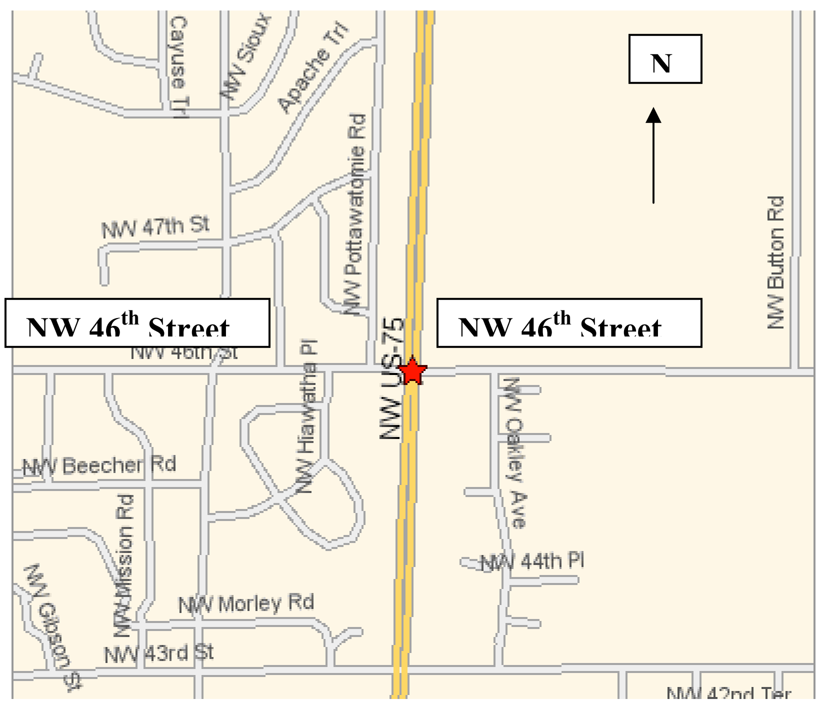


FIGURE 6.45: Figure Showing the Geographic Locations of US-75 and NW 46th Street Roundabout
Source: Adopted from Maps.Yahoo.com

- **Before Condition:** The intersection was signalized in the before condition.
- **Geometric parameters - Before condition:** In the before condition there were three approach lanes for the North Approach: One left turn lane and two through lanes. The South approach had four approach lanes: one right turn, one left turn and two through lanes. The East approach had three approach lanes. One lane was an exclusive left turn lane and one was a through lane. The other one was a through and right turn lane. There were two adjacent exit lanes. The West approach had three approach lanes. One lane was an exclusive left turn lane and one was a through lane. The other one was a through and right turn lane. All lanes were 12ft (3.7m) wide. The terrain was flat with zero gradients on all the approaches. See Figure 6.46 for the intersection in the before condition. See Figure 6.47 for the hourly turning movements.

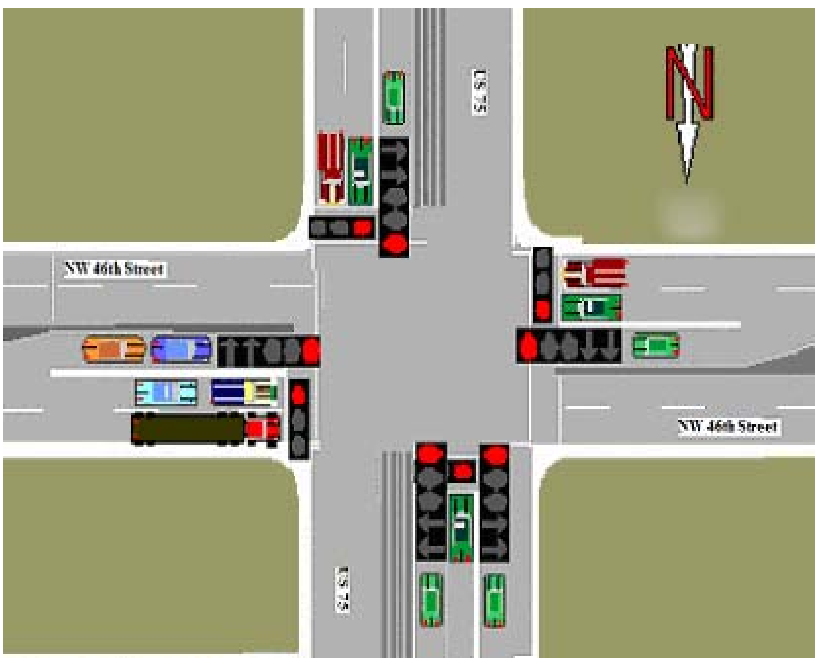


FIGURE 6.46: Figure Showing the Before Condition of US-75 and NW 46th Street

AM Condition

North

```
                       81        97        172
                      <--'        |        '-->
     138     ---^                 V                  ^---     152
West 402     --->                                    <---     315    East
      24     ---v                 ^                  v---      36
                      <--,        |        ,-->
                        7        122        66
```

South

PM Condition

North

```
                      174        185        170
                      <--'        |        '-->
      94     ---^                 V                  ^---     205
West 378     --->                                    <---     599    East
      13     ---v                 ^                  v---     111
                      <--,        |        ,-->
                       14        128        52
```

South

FIGURE 6.47: Hourly Turning Movements for US-75 & NW 46th Street in Before Condition (Traffic Signal)

- **After Condition:** In the after condition a modern roundabout was built. A new interchange was built with ramps going to and from US-75 onto the NW 46th street. The US-75 highway now goes over the roundabout. The roundabout has a well designed truck apron which handles the semi-trucks making and school buses that move in that area. The roundabout was designed as a two-lane roundabout but only one lane was operating during the study due to construction operation and has an oval central island. Key dimensions are given in the following section.

- **Geometric parameters - After condition:** The roundabout has a circulating lane width (W_c) of 16 feet (5m) on a flat terrain (zero gradient on all the approaches). During the study there was one circulating lane. The lane width is 12 feet (3.7 m). See Figures 6.48 for detailed drawing of the roundabout and 6.49 for artist sketch. See Figure 6.50 for hourly turning movements.

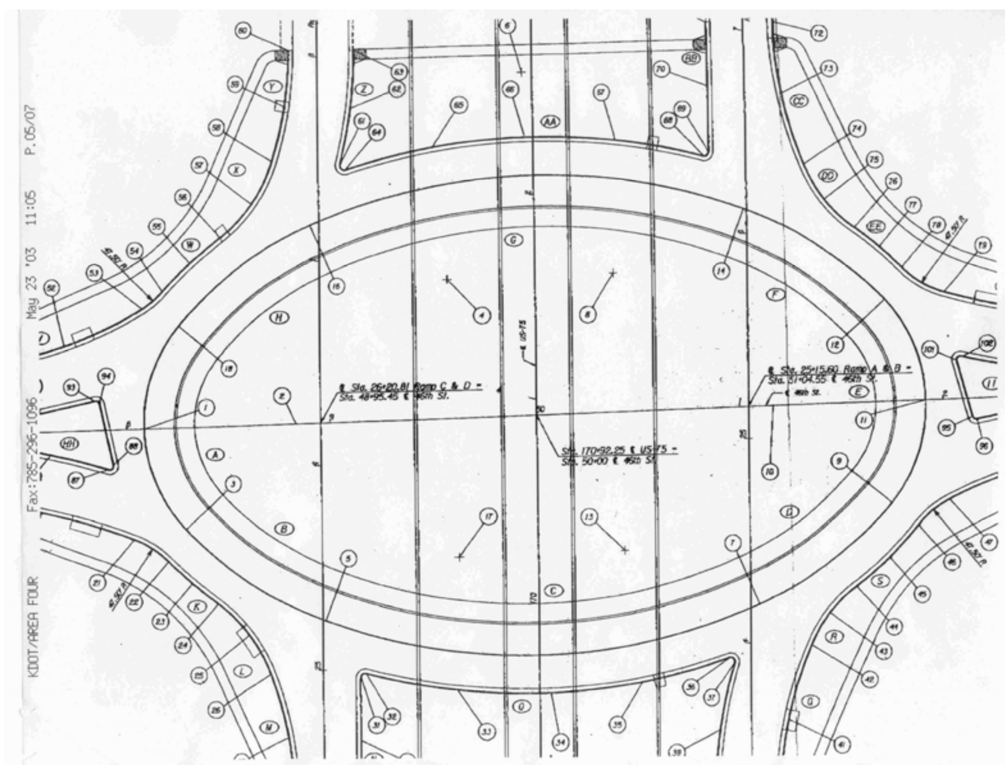


FIGURE 6.48: US-75 and 46th Street Intersection - After Condition
(Reduced from the actual plan provided by KDOT)

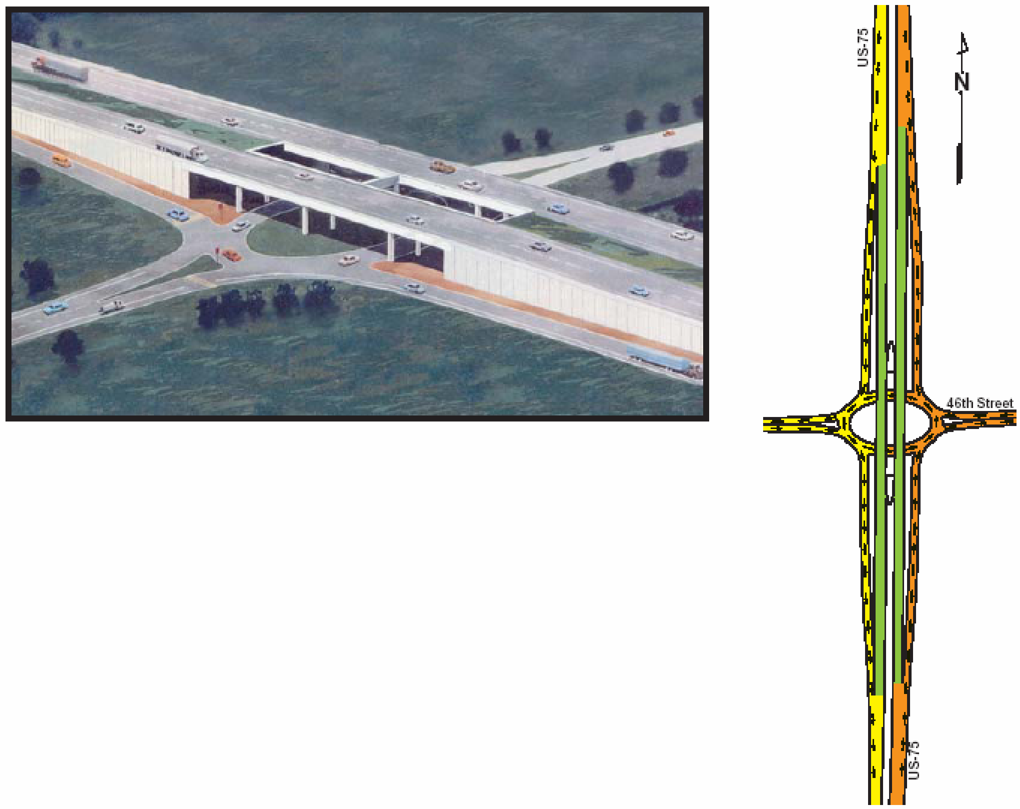


FIGURE 6.49: Figure Showing US-75 and NW 46th Street in After Condition
(Artist sketch provided by KDOT)

AM Condition

North

			135	**329**	**14**			
			<--'	**|**	**'-->**			
	64	**---^**		**V**		**^---**	**7**	
West	**265**	**--->**				**<---**	**442**	**East**
	25	**---v**		**^**		**v---**	**142**	
			<--,	**|**	**,-->**			
			115	**6**	**116**			

South

PM Condition

North

			281	**690**	**73**			
			<--'	**|**	**'-->**			
	55	**---^**		**V**		**^---**	**11**	
West	**281**	**--->**				**<---**	**438**	**East**
	124	**---v**		**^**		**v---**	**105**	
			<--,	**|**	**,-->**			
			56	**48**	**71**			

South

FIGURE 6.50: Hourly Turning Movements for US-75 and NW 46th Street in After Condition (Roundabout)

- **Statistics for AM and PM periods: US-75 and NW 46th Street:** Tables 6.17 and 6.18 give the statistics for the observed AM and PM period hourly traffic volumes, respectively, with the intersection operating under before (Signal) and after (RA) conditions and Figure 6.51 shows the traffic volume variation on a typical day with the intersection operating under each conditions.

TABLE 6.17: Descriptive Statistics for the AM Period Observed Hourly Traffic Volumes

	*Traffic Volumes Veh/hr**	
Statistics	***Intersection Treatments***	
	Signal	***RA***
Min	1158	1398
Mean	1462	1658
Max	1737	1980
Stdev	146	276
No. of Data points	10	6

*** Total Entering Vehicles**

TABLE 6.18: Descriptive Statistics for the PM Period Observed Hourly Traffic Volumes

	*Traffic Volumes Veh/hr**	
Statistics	***Intersection Treatments***	
	Signal	***RA***
Min	1692	1848
Mean	1976	2189
Max	2478	2385
Stdev	229	208
No.of Data points	10	6

*** Total Entering Vehicles**

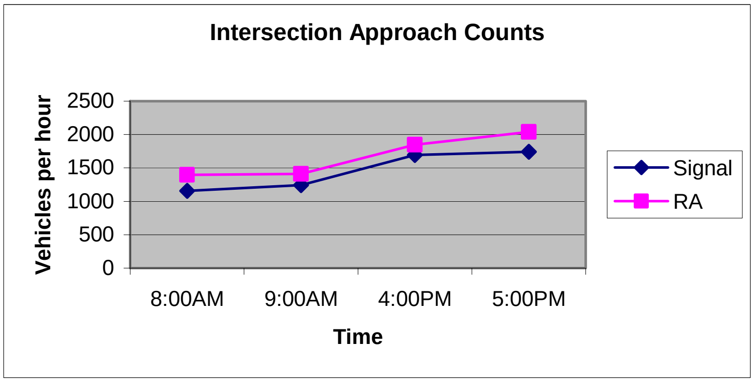


FIGURE 6.51: Traffic Volume Variation for a Typical Day

(Note: Lines in the figure are provided for reading convenience. No conclusions should be made that the lines indicate a statistical distribution or that there is a straight-line relationship between STIS time and vehicles per hour)

6.2.6 Hutchinson

The study intersection is the junction of two arterials in the center of a multi-functional area in the northeast section of Hutchinson, Kansas. The location, Severance Street and 23rd Avenue is shown in Figure 6.52. The information presented here is taken from an earlier study. [Sathya 2002]

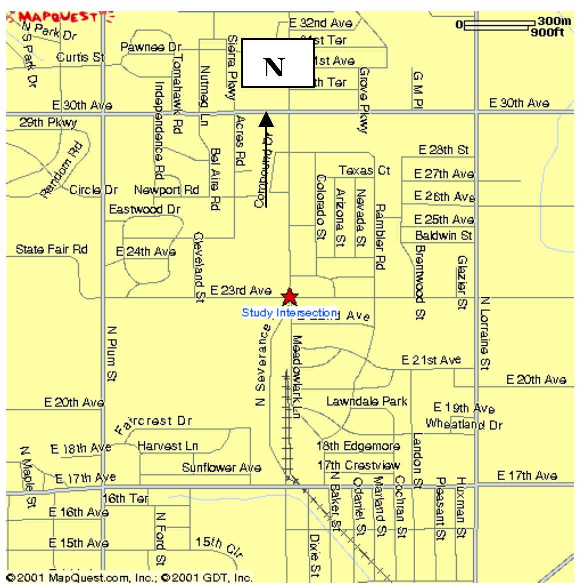


FIGURE 6.52: Figure Showing the Geographic Locations of Hutchinson Roundabout
Source: Adopted from MapQuest.com

- **Before Condition:** Prior to the installation of the modern roundabout at this site the intersection was controlled by stop signs on two approaches (Two Way Stop Control-TWSC). This type of control allows priority for the major street users and the minor street users wait for an acceptable gap to make the maneuver through the intersection. Intuitively, TWSC causes more delays for the minor street users and little or none for the major street users. Since the maneuvering from the minor street through the major street depends on subjective judgment of an acceptable gap, it creates a safety issue for the traffic entering the intersection.
- **Geometric parameters - Before condition:** Severance Street (N-S) has lane widths of 4.2 m (14 feet) and a 10.5 m (35 feet) median (drainage ditch) on both the approaches (see Figure 4.1). The major street, 23rd Avenue (E-W) has 3.6 m (12 feet) lane widths on the 23rd Avenue (E-W) with no median (see Figure 6.53). All the approaches are single lane. The posted speed limit is 48 km/h (30 mph) an all the approaches. Spot speeds obtained during a site visit indicated that the operating speeds on the approaches was 52 km/h (32 mph) on 23rd Avenue (E-W) and 61 km/h (38 mph) on Severance Street (N-S).

FIGURE 6.53 Two-Way Stop Control at 23rd Avenue and Severance Street

(View from South Approach) (Photo courtesy: Dr. Russell)

- **After Condition:** In the after condition a modern roundabout controls traffic. The roundabout built is a one-lane roundabout with a well designed truck apron and has an oval central island. Key dimensions are given in the following section.
- **Geometric parameters - After condition:** This first modern roundabout in Hutchinson, Kansas is a single lane roundabout with an oval central island. The roundabout has a circulating lane width (W_c) of 5.7 m (19 feet) on a flat terrain (zero gradient on all the approaches). The approach lane width is 3 m (10 feet) and 2.7 m (9 feet) for the N-S and E-W approaches respectively. The inscribed diameter (D_i) was measured to the middle of the stop line of the approach road in order to get the equivalent central island diameter measure for oval roundabouts. [Alcelik and Besley 1998]. The central island diameter was thus calculated as 30 m (100 feet) and used as input into SIDRA. The posted speed limits on the approaches were 48 km/h (30 mph). See Table 6.19 for hourly volume statistics.
- **Hourly volumes:** The traffic volume data was collected from 7:00AM to 1:00PM and from 1:00PM to 7:00PM on normal week days for the before and after conditions. The turning movement counts were obtained for every 15-minute interval and recorded.

TABLE 6.19: Descriptive Statistics for the Observed Hourly Traffic Volumes

	Traffic Volumes Veh/hr*	
Statistics	***Intersection Treatments***	
	TWSC	***RA***
Min	244	280
Mean	785	731
Max	1206	1110
Stdev	212	193
No. of Data points	153	46

*** Total Entering Vehicles**

6.2.7 Manhattan

The study intersection is the junction of two collector roads, Gary Avenue and Candlewood Drive in Manhattan, Kansas. The location is adjacent to a residential area. The geographic location is shown in Figure 6.54. The information presented here is taken from an earlier study. [Russell et.al., 2000]

FIGURE 6.54: Figure Showing the Geographic Locations of Manhattan Roundabout
Source: Adopted from Maps.Yahoo.com

- **Before Condition:** The before condition was a two-way stop controlled intersection. The intersection was not videotaped in before condition and hence a comparable intersection was considered for the purpose of analysis. The intersection considered was Dickens Avenue and Wreath Avenue.
- **Geometric parameters - Before condition:** The two roads are both two-lane with one lane in each direction. Parking is restricted near the intersection allowing creation of a turn lane on each approach. The north and south approaches have one left turn lane and a combined thru/right lane. The east and west approaches are stop controlled. The approach speeds ranged from 35 to 51 km/hr (22-32 mph).

- **After Condition:** In the after condition a single-lane modern roundabout traffic.
- **Geometric parameters - After condition:** The modern roundabout is a single lane roundabout with a circular central island. The approach lane widths are generally 4.6 m (15 feet). The central island is 9.1 meters (30 feet). See Table 6.20 for hourly volume statistics.
- **Hourly volumes:** The traffic volume data was collected from 7:00AM to 1:00PM and from 1:00PM to 7:00PM on normal week days for the after conditions. The turning movement counts were obtained for every 15-minute interval and recorded.

TABLE 6.20: Descriptive Statistics for the Observed Hourly Traffic Volumes

	Traffic Volumes Veh/hr*	
Statistics	***Intersection Treatments***	
	TWSC	***RA***
Min	215	224
Mean	444	387
Max	480	402

*** Total Entering Vehicles**

Chapter 7

Summary of Kansas Roundabout Results

7.1 General

The statistical analysis of the MOEs helps determine if and how the Stop controlled Intersections, Signal controlled Intersections and the Roundabout controlled Intersections differed in operation. The analysis provides information to assess characteristics of the Stop Controls, Traffic Signals and the Roundabout. The statistical testing was performed, as discussed in Chapter 3, separately for the AM and PM periods for all the locations in order to evaluate the operation of the intersection during these separate periods. The overall results of statistical testing for each location follow in Chapter 8.

7.2 Results for Kansas Roundabouts

A summary of combined results for the sites covered in this report is presented in this chapter. Table 7.1 gives the average values of all the sites studied. The after condition for all sites is a modern roundabout. The before conditions vary from site to site. Some have a Two-Way Stop Control, some have All-Way Stop Control and some have a Traffic Signal. Table 7.1 has been presented here to give an overall picture of roundabout performance in the state of Kansas. Table 7.2 gives average results for sites having All-Way Stop Control in their before condition. Table 7.3 gives average results for sites having Two-Way Stop Control in their before condition. Table 7.4 gives result for sites having Signal in before condition. Figures 7.1 through 7.3 give a graphical representation of results presented in Table 7.1. Individual results for each site are presented in the Appendix.

TABLE 7.1: Results for Kansas Roundabouts General (AM and PM combined)

All Kansas Sites Average				
Measures Of Effectiveness	**Before**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	20.2	8.0	-65%	Yes
Maximum Approach Delay (Seconds/veh)	34.4	10.4	-71%	Yes
95% Queue Length (Feet)	190	104	-44%	Yes
Degree Of Saturation V/C (Intersection)	0.463	0.223	-53%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	58	29	-52%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	62	37	-42%	Yes

Before: AWSC, TWSC, Signal **R.A: Roundabout**

Sites Included in the Kansas Average (11 sites in all) and before condition intersection control used in analysis:

Olathe: Ridgeview/Sheridan, Rogers/Sheridan (Before condition: AWSC) [2 sites]
Topeka: Rice Road North and South (Before condition: Theoretical TWSC) [2 sites]
: US-75/NW 46th Street (Before condition: Traffic Signal) [1 site]
Newton: I-135/Broadway, I-135/First Street (Before condition: Theoretical Traffic Signal) [2 sites]
Lawrence: Harvard Road/Monterey Way (Before condition: AWSC) [1 site]
Paola: Old K.C road/K-68 (Before condition: AWSC) [1 site]
Manhattan: Gary/Candlewood (Before condition: TWSC) [1 site]
Hutchinson: 23rd street/Severance Avenue (Before condition: TWSC) [1 site]

TABLE 7.2: Results for Locations with All-Way Stop Control in Before Condition

All-Way Stop control/ Roundabout Sites Average				
AM Results				
Measures Of Effectiveness	**AWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	28.4	8.4	-70%	Yes
Maximum Approach Delay (Seconds/veh)	44.6	10.5	-77%	Yes
95% Queue Length (Feet)	212.3	65.8	-69%	Yes
Degree Of Saturation V/C (Intersection)	0.700	0.225	-68%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	93	35	-63%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	90	35	-61%	Yes
PM Results				
Measures Of Effectiveness	**AWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	48.0	8.8	-82%	Yes
Maximum Approach Delay (Seconds/veh)	87.4	11.0	-87%	Yes
95% Queue Length (Feet)	481.5	85.9	-82%	Yes
Degree Of Saturation V/C (Intersection)	0.882	0.268	-70%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	93	40	-57%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	92	42	-54%	Yes

TABLE 7.3: Results for Locations with Two-Way Stop Control in Before Condition

Two-Way Stop control/ Roundabout Sites Average				
AM Results				
Measures Of Effectiveness	**AWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	7.1	4.2	-40%	Yes
Maximum Approach Delay (Seconds/veh)	13.6	5.3	-61%	Yes
95% Queue Length (Feet)	33.3	13.5	-59%	Yes
Degree Of Saturation V/C (Intersection)	0.222	0.110	-50%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	23	15	-35%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	41	29	-28%	Yes
PM Results				
Measures Of Effectiveness	**AWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	6.5	4.5	-30%	Yes
Maximum Approach Delay (Seconds/veh)	11.2	5.3	-53%	Yes
95% Queue Length (Feet)	38.5	13.0	-66%	Yes
Degree Of Saturation V/C (Intersection)	0.226	0.103	-54%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	25	16	-38%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	38	29	-23%	Yes

* Note: Manhattan and Hutchinson Sites Excluded

TABLE 7.4: Results for Locations with Traffic Signal in Before Condition

Traffic Signal/ Roundabout Sites Average				
AM Results				
Measures Of Effectiveness	Signal	R.A	% Diff.	Statistically Different
Average Intersection Delay (Seconds/veh)	21.4	7.6	-65%	Yes
Maximum Approach Delay (Seconds/veh)	35.1	10.1	-71%	Yes
95% Queue Length (Feet)	205.2	114.9	-44%	Yes
Degree Of Saturation V/C (Intersection)	0.489	0.227	-53%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	60	29	-52%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	63	36	-42%	Yes
PM Results				
Measures Of Effectiveness	Signal	R.A	% Diff.	Statistically Different
Average Intersection Delay (Seconds/veh)	29.0	17.9	-38%	Yes
Maximum Approach Delay (Seconds/veh)	41.3	27.0	-35%	Yes
95% Queue Length (Feet)	378.7	523.3	38%	No
Degree Of Saturation V/C (Intersection)	0.648	0.515	-21%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	81	51	-37%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	86	67	-22%	Yes

* Note: 95% queue length the results were not statistically different. Statistical testing of all data sets yielded this result. % Difference is not a measure of statistical differnce

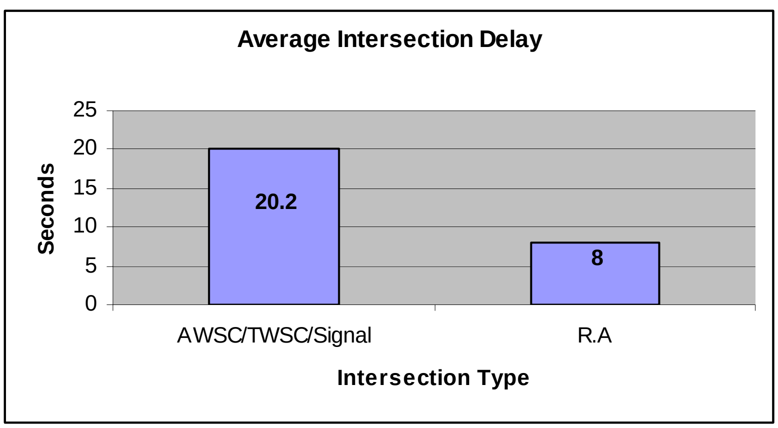


FIGURE 7.1: Figure Showing Comparison of Average Intersection Delay for all Kansas Sites

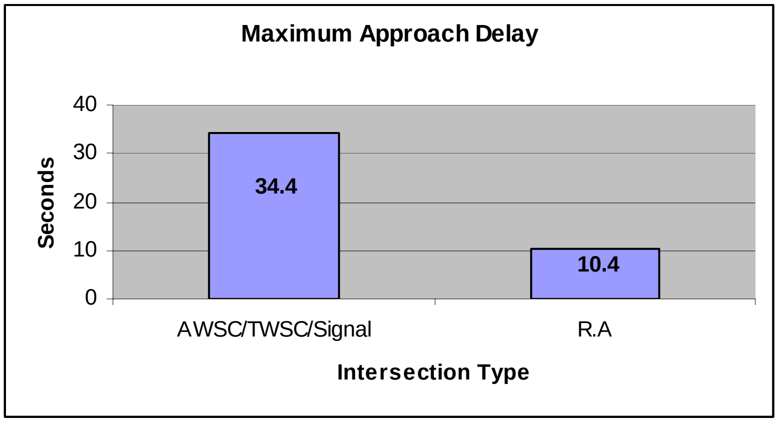


FIGURE 7.2: Figure Showing Comparison of Maximum Approach Delay for all Kansas Sites

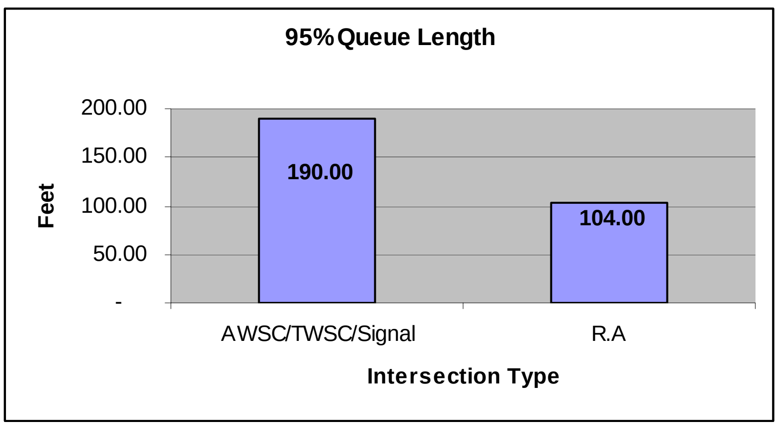


FIGURE 7.3: Figure Showing Comparison of 95% Queue Lengths for all Kansas Sites

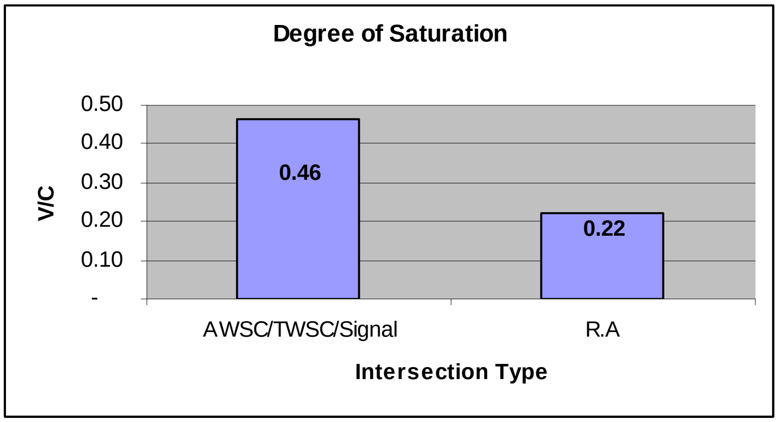


FIGURE 7.4: Figure Showing Comparison of Degree of Saturation for all Kansas Sites

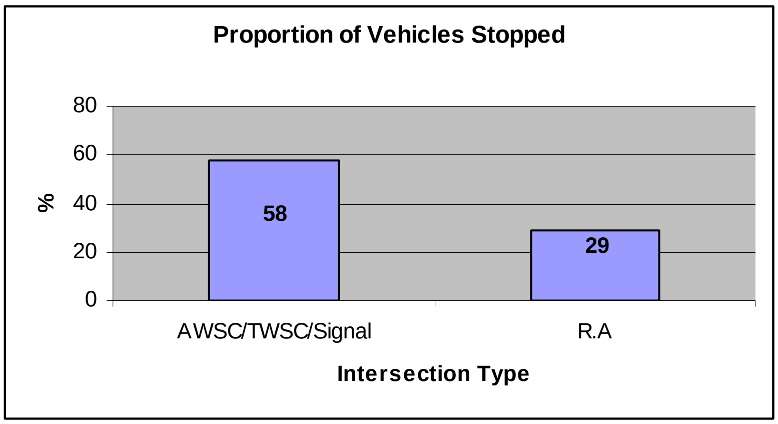


FIGURE 7.5: Figure Showing Comparison of Proportion of Vehicles Stopped for all Kansas Sites

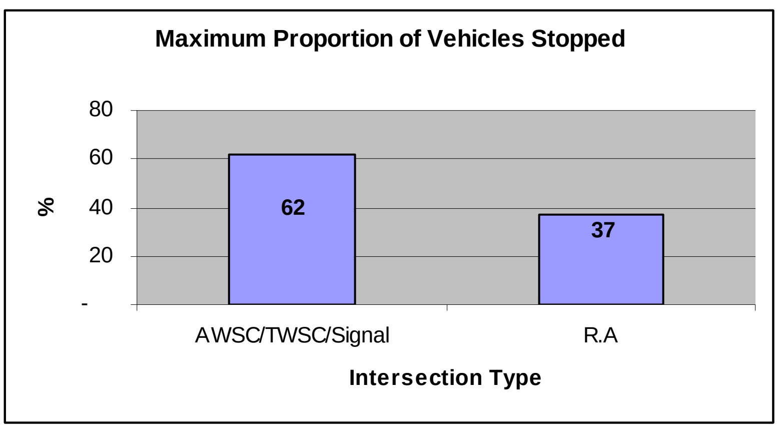


FIGURE 7.6: Figure Showing Comparison of Maximum Proportion of Vehicles Stopped for all Kansas Sites

7.3 Summary of Results for Kansas Roundabouts

- The Average Intersection Delay and Maximum Approach Delay are 65% and 71% less in the case of a modern roundabout. Since the delays experienced by vehicles are less in the case of a modern roundabout when compared to AWSC/TWSC/Signal, the intersection performance was enhanced.
- The 95% Queue Length is 44% less in the case of a modern roundabout. Since the queuing is directly proportional to delay the roadway efficiency is enhanced.
- The Degree Of Saturation is 53% less in the case of a modern roundabout. Since the v/c ratio can be a surrogate for Level Of Service (LOS) and is less in the case of a modern roundabout, the capacity was enhanced.
- The Proportion Of Vehicles Stopped and Maximum Proportion Of Vehicles Stopped are 52% and 42%, less respectively, in the case of a modern roundabout. Since the percentage of vehicles stopped is less in the case of a modern roundabout, and are related to queuing and delay, the intersection performance was enhanced.

Chapter 8

Conclusion

8.1 General

This chapter presents specific conclusions for Kansas Roundabouts based on the analysis of all the sites studied.

8.2 Conclusions about Kansas Roundabouts

- The modern roundabouts in Kansas operated more efficiently than the before intersection control (AWSC/TWSC/Signal) at all locations studied.
- There was a (65%) decrease in the Average Intersection Delay (Seconds/Vehicle) for the AM and PM periods combined, in the after condition after the installation of modern roundabout. The decrease was observed to be statistically significant.
- There was a (71%) decrease in the Maximum Approach Delay (Seconds/Vehicle) for the AM and PM periods combined, in the after condition after the installation of modern roundabout. The decrease was observed to be statistically significant.
- There was a (44%) decrease in the 95% Queue Length (feet) for the AM and PM periods combined, in the after condition after the installation of modern roundabout. The decrease was observed to be statistically significant.
- There was a (53%) decrease in the Degree of Saturation (v/c) for the PM and AM periods combined, in the after condition after the installation of modern roundabout. The decrease was observed to be statistically significant.
- There was a (52%) decrease in the Proportion of Vehicles Stopped (%) for the PM and AM periods combined, in the after condition after the installation of modern roundabout. The decrease was observed to be statistically significant.
- There was a (42%) decrease in the Maximum Proportion of Vehicles Stopped (%) for the PM and AM periods combined, in the after condition after the installation of modern roundabout. The decrease was observed to be statistically significant.

- Since the reductions in delay, queuing and proportion of vehicles stopped are statistically significant for the after condition of a modern roundabout, the movement of traffic through these intersections i.e., operational efficiency, should be significantly improved.
- Since all the locations had a range of different traffic conditions, it is reasonable to suggest that a modern roundabout may be the best intersection alternative for several other locations in Kansas with similar ranges of traffic volumes.
- Further studies should be conducted in other locations in Kansas with different traffic conditions, particularly those where volumes are high enough that a multi-lane roundabout is operating near capacity, in order to get a much clearer picture.

8.3 Overall Conclusion

Considering the above summary, it is concluded that the modern roundabouts studied significantly improved the operational efficiency of all intersections studied.

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Appendix

A.1 General

This chapter gives the results for each of the individual sites that have been studied.

OLATHE Location A: Olathe: Ridgeview and Sheridan

TABLE A.1: Results for Olathe: Ridgeview and Sheridan

Olathe:Ridgeview & Sheridan				
AM Results				
Measures Of Effectiveness	**AWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	46.1	10	-78%	No
Maximum Approach Delay (Seconds/veh)	66	13	-80%	Yes
95% Queue Length (Feet)	402	73	-82%	Yes
Degree Of Saturation V/C (Intersection)	0.98	0.27	-72%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	94	31	-67%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	100	52	-48%	Yes
PM Results				
Measures Of Effectiveness	**AWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	66.5	11.0	-83%	Yes
Maximum Approach Delay (Seconds/veh)	118.5	15.1	-87%	Yes
95% Queue Length (Feet)	642	152	-76%	Yes
Degree Of Saturation V/C (Intersection)	1.16	0.43	-63%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	94	41	-56%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	100	64	-36%	Yes

OLATHE Location B: Olathe: Rogers and Sheridan

TABLE A.2: Results for Olathe: Rogers and Sheridan

Olathe: Rogers&Sheridan				
AM Results				
Measures Of Effectiveness	**AWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	37.6	11.7	-69%	Yes
Maximum Approach Delay (Seconds/veh)	65.7	15.5	-76%	Yes
95% Queue Length (Feet)	333	149	-55%	Yes
Degree Of Saturation V/C (Intersection)	0.95	0.4	-58%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	100	65	-35%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	88	46	-48%	Yes
PM Results				
Measures Of Effectiveness	**AWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	90.4	11.9	-87%	Yes
Maximum Approach Delay (Seconds/veh)	164.2	15	-91%	Yes
95% Queue Length (Feet)	1125	147	-87%	Yes
Degree Of Saturation V/C (Intersection)	1.29	0.37	-71%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	100	63	-37%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	97	51	-47%	Yes

Location 2: Lawrence Results

TABLE A.3: Results for Lawrence

Lawrence				
AM Results				
Measures Of Effectiveness	**AWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	12.7	6.4	-50%	Yes
Maximum Approach Delay (Seconds/veh)	16.6	7	-58%	Yes
95% Queue Length (Feet)	37	21	-43%	Yes
Degree Of Saturation V/C (Intersection)	0.4	0.1	-75%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	80	10	-88%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	90	20	-78%	Yes
PM Results				
Measures Of Effectiveness	**AWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	13.6	6.6	-51%	Yes
Maximum Approach Delay (Seconds/veh)	15.9	7.2	-55%	Yes
95% Queue Length (Feet)	54	23	-57%	Yes
Degree Of Saturation V/C (Intersection)	0.5	0.1	-80%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	80	20	-75%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	90	30	-67%	Yes

Location 3: Paola Results

TABLE A.4: Results for Paola

Paola				
AM Results				
Measures Of Effectiveness	**AWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	17	5.4	-68%	Yes
Maximum Approach Delay (Seconds/veh)	30	6.3	-79%	Yes
95% Queue Length (Feet)	77	20	-74%	Yes
Degree Of Saturation V/C (Intersection)	0.47	0.13	-72%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	98	32	-67%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	80	21	-74%	Yes
PM Results				
Measures Of Effectiveness	**AWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	21.5	5.6	-74%	Yes
Maximum Approach Delay (Seconds/veh)	51	6.6	-87%	Yes
95% Queue Length (Feet)	105	22	-79%	Yes
Degree Of Saturation V/C (Intersection)	0.58	0.17	-71%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	99	37	-63%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	80	23	-71%	Yes

TOPEKA Location 1a: Rice Road North Results

TABLE A.5: Results for Rice Road North Roundabout

Rice Road North Roundabout				
AM Results				
Measures Of Effectiveness	**TWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	9.2	3.8	-59%	Yes
Maximum Approach Delay (Seconds/veh)	16.3	5.1	-69%	Yes
95% Queue Length (Feet)	53.5	22	-59%	Yes
Degree Of Saturation V/C (Intersection)	0.354	0.172	-51%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	28	18	-36%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	50	39	-22%	Yes
PM Results				
Measures Of Effectiveness	**TWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	5.4	3.8	-30%	Yes
Maximum Approach Delay (Seconds/veh)	10.2	4.3	-58%	Yes
95% Queue Length (Feet)	11	6	-45%	Yes
Degree Of Saturation V/C (Intersection)	0.086	0.046	-47%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	17	13	-24%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	28	20	-29%	Yes

TOPEKA Location 1b: Rice Road South Results

TABLE A.6: Results for Rice Road South Roundabout

Rice Road South Roundabout				
AM Results				
Measures Of Effectiveness	**TWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	4.9	4.6	-6%	No
Maximum Approach Delay (Seconds/veh)	10.9	5.5	-50%	Yes
95% Queue Length (Feet)	13	5	-62%	Yes
Degree Of Saturation V/C (Intersection)	0.089	0.048	-46%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	18	12	-33%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	31	19	-39%	Yes
PM Results				
Measures Of Effectiveness	**TWSC**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	7.5	5.2	-31%	Yes
Maximum Approach Delay (Seconds/veh)	12.2	6.3	-48%	Yes
95% Queue Length (Feet)	66	20	-70%	Yes
Degree Of Saturation V/C (Intersection)	0.365	0.16	-56%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	33	18	-45%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	47	38	-19%	Yes

TOPEKA Location 2: US75/NW 46th Street Results

TABLE A.7: Results for US75/NW 46th Street Roundabout

US 75 and NW 46th Street Roundabout				
AM Results				
Measures Of Effectiveness	SIGNAL	R.A	% Diff.	Statistically Different
Average Intersection Delay (Seconds/veh)	32.44	8.48	-74%	Yes
Maximum Approach Delay (Seconds/veh)	47.35	13.85	-71%	Yes
95% Queue Length (Feet)	302	207	-31%	No
Degree Of Saturation V/C (Intersection)	0.587	0.574	-2%	No
Proportion Of Vehicles Stopped (%) (Intersection)	84	55	-35%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	88	81	-8%	No
PM Results				
Measures Of Effectiveness	SIGNAL	R.A	% Diff.	Statistically Different
Average Intersection Delay (Seconds/veh)	67.62	34.51	-49%	Yes
Maximum Approach Delay (Seconds/veh)	98.49	60.15	-39%	No
95% Queue Length (Feet)	956	1478	55%	No
Degree Of Saturation V/C (Intersection)	0.908	1.05	16%	No
Proportion Of Vehicles Stopped (%) (Intersection)	89	77	-13%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	95	100	5%	Yes

NEWTON Location 1: I-135 and First Street Results

TABLE A.8: Results for Newton: I135 and First Street Roundabout

Newton I135 & First Street				
AM Results				
Measures Of Effectiveness	**SIGNAL**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	8.1	8	-1%	No
Maximum Approach Delay (Seconds/veh)	9.5	8.4	-12%	Yes
95% Queue Length (Feet)	54	40	-26%	Yes
Degree Of Saturation V/C (Intersection)	0.422	0.213	-50%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	74	35	-53%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	79	44	-44%	Yes
PM Results				
Measures Of Effectiveness	**SIGNAL**	**R.A**	**% Diff.**	**Statistically Different**
Average Intersection Delay (Seconds/veh)	9.1	8.2	-10%	Yes
Maximum Approach Delay (Seconds/veh)	10.6	8.7	-18%	Yes
95% Queue Length (Feet)	90	47	-48%	Yes
Degree Of Saturation V/C (Intersection)	0.514	0.25	-51%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	77	40	-48%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	82	47	-43%	Yes

NEWTON Location 2: I-135 and Broadway Results

TABLE A.9: Results for Newton: I-135 and Broadway Roundabout

Newton I135 & Broadway				
AM Results				
Measures Of Effectiveness	SIGNAL	R.A	% Diff.	Statistically Different
Average Intersection Delay (Seconds/veh)	10.2	10.4	2%	No
Maximum Approach Delay (Seconds/veh)	14.5	12	-17%	Yes
95% Queue Length (Feet)	103	52	-50%	Yes
Degree Of Saturation V/C (Intersection)	0.58	0.275	-53%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	72	30	-58%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	77	48	-38%	Yes
PM Results				
Measures Of Effectiveness	SIGNAL	R.A	% Diff.	Statistically Different
Average Intersection Delay (Seconds/veh)	10.4	11	6%	No
Maximum Approach Delay (Seconds/veh)	14.9	12.1	-19%	Yes
95% Queue Length (Feet)	90	45	-50%	Yes
Degree Of Saturation V/C (Intersection)	0.523	0.244	-53%	Yes
Proportion Of Vehicles Stopped (%) (Intersection)	78	37	-53%	Yes
Max.Proportion Of Vehicles Stopped (%) (Approach)	81	54	-33%	Yes