

# **Do Roundabouts Work? An Evaluation for Uniform Approach Demands**

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Thesis submitted to the faculty of the Virginia Polytechnic Institute and State University in partial fulfillment of the requirements for the degree of

Master of Science  
In  
Civil Engineering

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August 1, 2011  
Blacksburg, VA

Keywords: Roundabouts, Emissions modeling, Delay

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## **Abstract**

With the increased prevalence of roundabouts in the United States, there is a need to evaluate the performance of roundabouts relative to other intersection control strategies. Few studies have compared roundabouts with other intersection control strategies in a systematic fashion. Consequently, this thesis compares four types of intersection control strategies considering a single lane approach with a 35 mph speed limit and equal demand on all approaches. The study demonstrates that vehicle delay is minimized with the use of a roundabout intersection control for all demand levels below 500 veh/hr/approach. Above this point if the left turn percentage exceeds 70%, traffic signal control is more efficient. The roundabout alternative also produces the fewest vehicle stops for low demand levels, low left turn demand and high right turn demand; however a two- way stop control alternative produces the least number of vehicle stops when the through and total demand is high. This study illustrates that fuel consumption and carbon dioxide, carbon monoxide, hydrocarbon and nitrogen oxide emissions can be improved with roundabout control over other intersection control strategies. The research presented here demonstrates that for low traffic demand levels roundabouts should be part of design alternatives considered for isolated intersection control.

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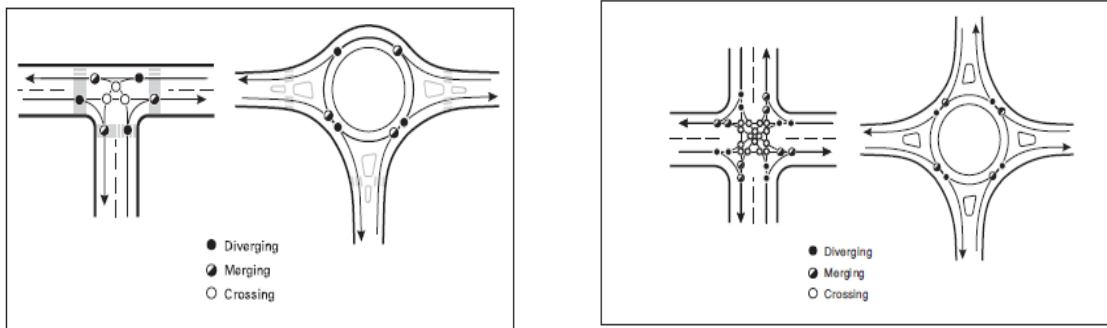
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# Chapter 1: Introduction

In 1966, the modern roundabout was developed in England to combat the growing issues of safety at circular intersections. Previously these intersections, known as rotaries, gave priority to entering traffic and had high speed approaches which led to high congestion and high crash rates. Unlike rotaries or traffic circles that are mainly used for traffic calming and have either no right of way condition or a stop at all approaches, in a modern roundabout priority is given to circulating vehicles leading to an improvement in safety. In 1990 the first roundabout was implemented in the United States (US) [1].

As roundabouts have become more prevalent especially in the last decade, the Federal Highway Administration (FHWA) in conjunction with Kittelson and Associates have researched and published “Roundabouts: An Informational Guide” in 2000 with an update in 2010. This guide helped to define a roundabout as a circulatory intersection with yield to circulating traffic, channelized approaches and geometries that facilitate reduction of speeds to 30 mph or less [1,2].

These guides also suggest reasons for the increased number of roundabout implementations in the US. Under normal operating conditions, roundabouts can improve safety due to fewer conflict points and lower, more uniform speeds. In Figure 1-1, a comparison of a roundabout to a traditional intersection is made. For a three-legged or T-intersection, a single-lane roundabout has three merging and three diverging conflict points while a traditional intersection has three merging, three diverging and three crossing conflict points. For a four-legged approach a single-lane roundabout has four merging and four diverging conflict points while a traditional intersection has 4 merging, four diverging and 16 crossing conflict points [2]. With these additional conflict points a traditional intersection is inherently more dangerous. In addition to the fewer conflict points, a geometry that slows traffic to a more uniform speed (as is the case with a roundabout) can be seen as safer.



a. Three-legged intersection

b. Four-Legged intersection

Figure 1-1: One lane intersection conflict points under normal operating conditions [3]

There are other factors that must be considered before building a roundabout in place of a traditional intersection. One of these is the space requirements for the roundabout. Unless one considers building a mini-roundabout, roundabouts require more real estate compared to traditional intersections. However, a traditional intersection can require more space on approaches in order to accommodate turning lanes [1]. The geometry determines speed and creates a geometric delay. Another factor to consider is heavy vehicles which both decrease capacity and therefore increase delay as well as building considerations. To properly make turns heavy vehicles need at least a mountable apron or a fully mountable island [1]. Considering

turning demands also needs to be factored in the decision process. Krogscsheepers and Roebuck suggest that high turning demands especially for left turning traffic creates a higher delay [3].

Finally, education and public opinion must be considered. Public opinion of roundabouts is often poor prior to a roundabout being built in an area. This may be due to familiarity with traditional intersections versus a lack of knowledge about roundabouts. This can be ameliorated through education before a roundabout is built or through proper signage so that those unfamiliar with the area can still safely navigate the intersection [4].

### 1.1 Problem Definition

As compared to traditional intersections, a new way of quantifying delay and comparing the efficiency of roundabouts is needed to truly discuss when to use a roundabout over other forms of intersection control. The Highway Capacity Manual 2000 (HCM 2000) recommends ranges of demands for a major and minor street in which two-way stop, all-way stop, and signalized intersection control are warranted (see Figure 1-2). For low minor demands a two way stop control (TWSC) is recommended while at high minor street demands a traffic signal is more efficient. For moderate demands in both directions an all way stop control (AWSC) is recommended. However, the figure does not include the roundabout as one of the isolated signalized control options and suggests that they may be a “viable alternative for portions of these ranges.”[5] Unfortunately, the measures of effectiveness used in assessing the optimum isolated intersection control in the figure are not documented. Potential measures of effectiveness include intersection capacity and delay.

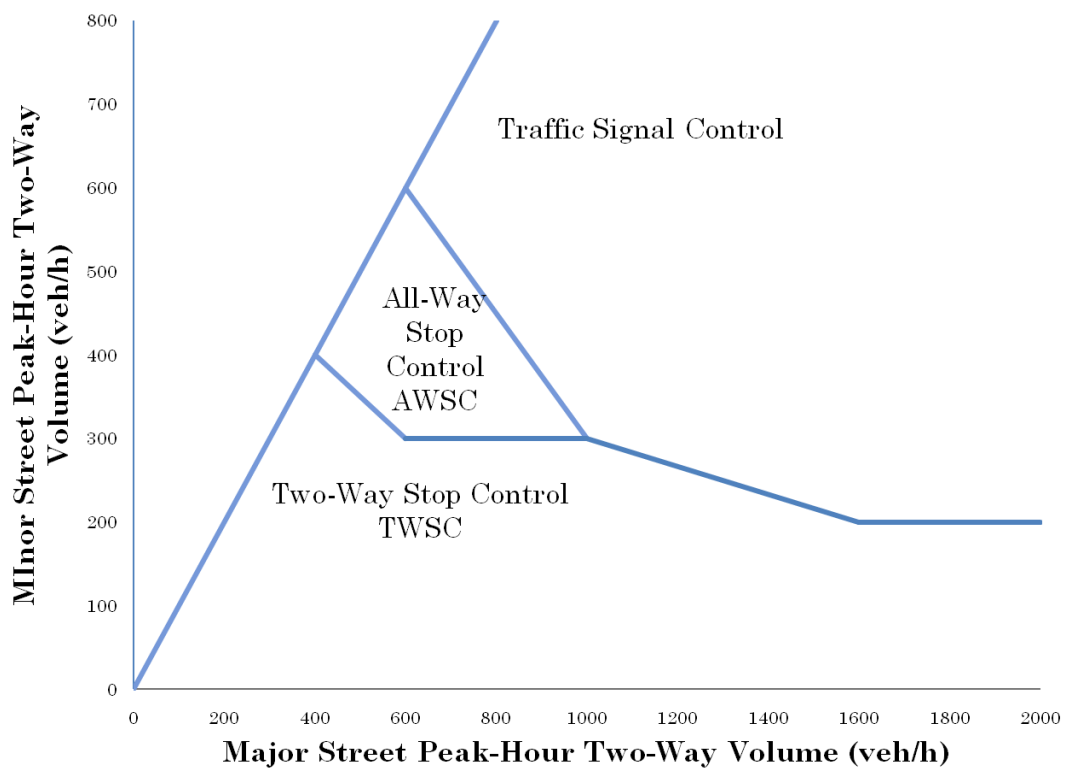


Figure 1-2: Volumes recommended for different traffic control devices. Roundabouts may be a viable alternative for portions of these ranges. [5]

In an exhaustive literature review, there have been many capacity and delay equations synthesized. Most are either linear or based on gap acceptance models. In other papers intersection controls were compared with both traditional measures of effectiveness (MOE) and environmental MOEs. Many of these studies have been case studies of a particular intersection and few studies have compared roundabouts with other intersection control strategies in a systematic fashion. Some studies have suggested that when using environmental measures of effectiveness roundabouts can have few emissions and lower fuel consumption when compared to unsignalized intersections and can be better for lower demands than signalized intersections. Consequently, this thesis compares four types of intersection control strategies considering a single lane approach with a 35 mph speed limit and equal demand on all approaches.

## **1.2 Research Objectives and Contributions**

This research aims to generalize a comparison of intersection controls for a non-specific isolated intersection with four single lane approaches. In the literature, intersections have been compared using either a very narrow range of turning demands or narrow range of total approach demands; these ranges are often associated with real world demands at a particular intersection. This research goes beyond the narrow focus of case studies to get a wider comparison of average delay, number of stops, fuel consumption and carbon dioxide, carbon monoxide, nitrogen oxides and hydrocarbon at different alternatives with equal demand on all approaches. These alternatives are a roundabout, an all-way stop, a two-way stop and a pre-timed signalized intersection.

## **1.3 Layout of Thesis**

This thesis is organized into five chapters. The literature review in the second chapter discusses current models for delay and capacity for roundabouts as well as many studies that compare and contrast roundabouts to other intersection control including all-way stop controls, two way stop controls and traffic signals. The third chapter presents the generalized intersection with four single-lane approaches and how the intersection controls' delay and number of stops compare. The fourth chapter continues research comparing nitrogen oxide, carbon monoxide, carbon dioxide and hydrocarbon emissions and fuel consumption. The final and fifth chapter concludes the study and recommends further research.

## **1.4 References**

1. Robinson, B. W., R. Lee, et al. (2000). Roundabouts: An Informational Guide, Kittelson & Associates and Federal Highway Administration: 277.
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4. Retting, R. A., S. Mandavilli, et al. (2006). Traffic Flow and Public Opinion: Newly Installed Roundabouts in New Hampshire, New York, and Washington. Transportation Research Board. Washington, D.C.: 10.
5. TRB Chapter 10: Urban Street Concepts. Highway Capacity Manual 2000. 2000. p. 10-21.



## Chapter 2: Literature Review

This chapter presents a literature review on roundabouts with an emphasis on environmental issues. In the search, papers were divided into four groups: capacity and delay, other design aspects, comparison of intersection control strategies using traditional measures of effectiveness (MOEs), and comparisons of intersections using environmental MOEs. This review provides some insight as to how to calculate the delay and capacity of a roundabout approach, what design factors may be important in identifying the optimum intersection control strategies, and how other studies compared and evaluated roundabout intersection control. When comparing a roundabout many of these studies used a base case from a particular intersection and discussed how that could be generalized; however some papers showed that these generalizations can be faulty.

### 2.1 Capacity and Delay

Of the four groups most publications fall in this category. This is due to the fact that most research efforts have been conducted on how to compute the capacity of roundabouts for the evaluation of the operation of existing and the design of new roundabouts. The US, namely FHWA and the Transportation Research Board, and a few states, including Pennsylvania, Arizona and Florida have created design guides to deal with how to calculate MOEs to compare a roundabout with other types of intersection control. Most papers propose control delay as the selected MOE for comparison purposes. In the US the delay equation is based on the delay at a stop controlled intersection. However due to the fact that at a roundabout a car does not have to come to a complete stop the equation has been modified to include a term for higher capacities that slow down the operation and do not include an absolute total delay that a person experiences regardless of whether or not there are conflicting vehicles [1]. This equation is as follows:

$$d = \frac{3600}{c_{m,x}} + 900T \times \left[ \left( \frac{v_x}{c_{m,x}} \right)^2 + \frac{\left( \frac{3600}{c_{m,x}} \right) \left( \frac{v_x}{c_{m,x}} \right)}{450T} \right] \quad (2-1)$$

Where  $d$ =average control delay (sec/veh)  
 $V_x$ =Flow Rate for movement  $x$  (veh/h)  
 $C_{m,x}$ =capacity of movement  $x$  (veh/h)  
 $T$ =analysis period (h)

[1 2]

The equation from the FHWA uses the capacity of an approach, however this capacity must be found through modeling. Currently there are many models that can estimate the capacity of a roundabout approach. There are a few similarities in the models. Most models are either exponential or linear, and either uses a gap acceptance model or the geometry of the roundabout. These are also broken down into single lane and double lane roundabouts.

The FHWA uses a linear regression model for both double and single lane roundabouts [2]:

$$Q_e = \begin{cases} K(F - f_c Q_c), & f_c Q_c \leq F \\ 0, & f_c Q_c > F \end{cases} \quad (2-2)$$

Where:  $Q_e$ =entry capacity (pce/h)  
 $Q_c$ =Circulating flow (pce/h)

$$k = 1 - 0.00347(\phi - 30) - 0.978 \left( \frac{1}{r} - .05 \right) \quad (2-3)$$

$$F = 303x_2$$

$$f_c = 0.210t_d(1 + 0.2x_2)$$

$$t_d = 1 + \frac{0.5}{1 + \exp\left(\frac{D-60}{10}\right)}$$

$$x_2 = v + \frac{e-v}{1+2S}$$

$$S = \frac{1.6(e-v)}{l'}$$

Where: e=Entry width (m)  
v=approach half width (m)  
l'=effective flare (m)  
S= sharpness  
D=inscribed circular diameter (m)  
φ= entry angle in degrees  
r=entry radius (m)

Other international linear regression models including the UK, Australian, German, French and Swiss models are of similar structure but use these and other geometrical parameters. HCM 2000 uses an exponential model similar to the gap acceptance capacity model [3].

$$c_a = \frac{v_c e^{-v_c t_c / 3600}}{1 - e^{-v_c t_f / 3600}} \quad (2-4)$$

Where: c<sub>a</sub>=Approach Capacity (veh/hr)  
v<sub>c</sub>=volume of conflicting traffic  
t<sub>c</sub>=critical gap(s)  
t<sub>f</sub>=follow up time(s)

HCM 2000 suggests using a t<sub>c</sub> value of 4.1 s and t<sub>f</sub> values of 2.6 s [3]. Other papers suggest different values summarized in Figure 2-1.

Model	T <sub>c</sub>	T <sub>f</sub>
HCM upper bound [3]	4.1	2.6
HCM lower bound [3]	4.6	3.1
Flannery and Datta [4]	3.45-5.11	2.20-2.82
Lindenmann[5]	2.80-3.96	1.88-2.83
Xie, Yan et al. [6]	5.0	3.0

Figure 2-1: Critical gap and follow-up gap times from different sources

Although these models are recommended by the United States many of the papers on roundabouts use the models in SIDRA. This software allows for easy comparison of different intersections but also has its own model for roundabouts. This model is based on the research conducted by Ackelik [1], as follows:

$$q_{e,max} = \max(f_{od}q_g, q_m)$$

$$q_g = \frac{3600}{\beta} \left(1 - \Delta \frac{q_c}{3600} + 0.5\beta\phi_c \frac{q_c}{3600}\right) \exp(-\lambda(\alpha - \Delta_c)) \quad (2-5)$$

$$q_m = \min(q_e, 60n_m)$$

$$f_{od} = 1 - f_{qc}(p_{qd}p_{cd})$$

Where q<sub>e,max</sub>=max entry flow (veh/h)  
q<sub>g</sub>= min entry flow (veh/h)  
q<sub>c</sub>=conflicting flow (veh/h)  
f<sub>od</sub>=origin destination adjustment factor  
p<sub>cd</sub>p<sub>qd</sub>=0.5 to 0.8

$n_m$  = min entry flow (veh/min)  
 $n_c$  = number of lanes conflicting  
 $\Delta_c$  = min headway in circulating traffic (2.00 for  $n_c=1$  and 1.2 for  $n_c=2$ )

$\lambda$  = arrival headway distribution factor(veh/s)

$$= \begin{cases} \frac{\varphi_c q_c / 3600}{1 - \Delta_c q_c / 3600} & \text{for } q_c / 3600 \leq 0.98 / \Delta_c \\ \frac{4.9 \varphi_c}{\Delta_c} & \text{else} \end{cases}$$

$\varphi_c$  = proportion of unbunched conflicting vehicles

$$= \exp(-5.0 q_c / 3600) \text{ for } n_c = 1$$

$$= \exp(-3.0 q_c / 3600) \text{ for } n_c = 2$$

$\beta$  = follow-up headways (s)

## 2.2 Other Design Parameters

Research suggests that these MOEs can be affected by geometric design and demands on the intersection. Most design guides suggest that the traffic must be approximately equal on all approaches [2]. A paper authored by Krogscheepers and Roebuck goes so far as suggest that turning percentages also effect the operations around an intersection [7]. Using SIDRA, a micro-simulation program called TRACSIM and demand data of between 1710 and 1830 veh/hr which synthesized from the micro-simulator, he suggests that turning percentages that are high create high delay. His results however are inconclusive for lower turning percentages. [7]

In most design guidelines, the discussion of geometric design centers around how fast a vehicle can go through the intersection. The FHWA and many states suggest that the radius of curvature for the entry to a roundabout should be greater than the radius of the circle and the exit. [2 8 9] This difference in curvature should allow for cars to slow down before the roundabouts and force safer speeds all the way through. The design guides also suggest that the center line of each approach should go through the center or to the left of center to allow for slower entry. [2 8]

## 2.3 Comparison of Intersection Control

In the survey of papers, seven papers dealt with the subject of comparing intersection controls. Most have come to a consensus on the “best” intersection design when comparing stopped or signalized controls to roundabouts. All papers suggest that as compared to any stop or signalized control, as long as the demands of the approaches are not unbalanced (one or two approaches having considerably more demand than the others), a roundabout will always be an equal or better choice. In one example, using dynamic modeling, Zhang et al. compared a pretimed signal to a roundabout and showed that for longer queues and therefore larger demand the signal will have lower delays but for shorter queues and therefore smaller demand a roundabout will have shorter delay. [10]

In a paper written by Russell, Rys et al., roundabouts were compared to two all way stop controlled intersections and two two-way stop controlled intersections. Using SIDRA, they demonstrated qualitatively that a roundabout when used in the proper situation, with even approach demands, speeds less than 25 mph, and proper geometric design, will perform better in most standard MOEs than an AWSC or TWSC intersection. Their results, summarized in Figure 2-2, show a roundabout will have shorter queue lengths, lower approach delays, smaller proportion of vehicles will stop, lower maximum approach vehicles will stop, and a lower degree of saturations as compared to the AWSC or TWSC intersections. The TWSC and roundabout have approximately the same average delay which is still less than either all-way stop controlled intersection. [11]

In a Kansas before and after study, the comparison demonstrated qualitatively that in these cases the roundabout was a viable option (see Figure 2-2) [11 12]. This was also shown in before and after quantitative analysis around three different roundabouts in New Hampshire, New York and Washington.

These roundabouts had a daily volume of 7,600 - 11,250 vehicles. They showed a 83% to 93% decrease in average intersection delay and a 31% to 87% reduction in total stops. [13]

Measure of Effectiveness:	Statistical Result: <sup>a</sup>	Traffic Control Advantage:
95% Queue	RA < 4L = 2S < 4S	Roundabout
Average Delay	RA = 2S < 4S < 4L	Roundabout/two-way stop
Maximum Approach Delay	RA < S2 < 4S < 4L	Roundabout
Proportion Stopped	RA < 2S < 4L < 4S	Roundabout
Maximum Approach Stopped	RA < 2S < 4L < 4S	Roundabout
Degree of Saturation	RA < 2S < 4S < 4L	Roundabout

<sup>a</sup> RA = roundabout; 2S = two-way stop; 4S = four-way stop and 4L = four-way stop with added turn lane

Figure 2-2: Qualitative measures of effectiveness from Kansas [11]

Other researchers, Sisioupiku and Oh, compared TWSC and AWSC, signal control, and yield to a Roundabout. They used volume ratio approaches that were even for all approaches, left turning percentages of 10%, 20% and 30% and assumed the defaults of the SIDRA program for the roundabout's geometry. In their comparison they demonstrated that when demand is light, two way stop and yield control can be highly effective. In most cases, all-way stop controlled intersections were revealed to have higher delay than either roundabouts or signalized intersections. Their research also demonstrated that roundabouts and signalized intersections have comparable delays when approaches were single-lane. double-lane approaches and large demand roundabouts have lower delays than traffic signals, and three-lane approach signal controls have lower delay than roundabouts. They demonstrated for heavy left turns two lane roundabouts have lower delay and increased capacity.[12]

## 2.4 Environmental Studies

Many studies have discussed the comparison of the six basic MOEs. Some of these same studies briefly describe the change of a traditional intersection to a roundabout and how this change impacts fuel consumption and emissions. One author, Garder, suggests that due to the reductions in stops and delay there should be an overall reduction in emissions and fuel consumption as long as there is a similar amount of traffic from the minor street when the intersection is designed to have 18 mph (29kmph).[15] He suggests that when comparing vehicle-actuated signal controls to roundabouts the actuated signal would have less fuel consumption and less emissions when traffic volumes are low but when demand is high and the actuated traffic signal functions more like a pre-timed traffic signal the percentage of cars that must stop increases and therefore the fuel consumption and emissions would improve with a roundabout. [15]

As many of the papers in this section use SIDRA for both the basic MOEs and environmental MOEs, the equations and tables follow for the SIDRA emissions model. [16]

For fuel consumptions and all emissions except CO<sub>2</sub> [16]:

$$\Delta F = \begin{cases} \{\alpha + \beta_1 R_T v + [\beta_2 M_v a^2 v / 1000]_{a>0}\} \Delta t & \text{for } R_t > 0 \\ \alpha \Delta t & \text{for } R_t \leq 0 \end{cases} \quad (2-6)$$

Where: R<sub>t</sub>=Tractive force (kN)

M<sub>v</sub>=Mass (kg)

v=speed (instantaneous) =v (km/h)/3.6

a=instantaneous acceleration (m/s<sup>2</sup>)

$\alpha$ =idle fuel rate or emissions rate

$\beta_1$ = efficiency parameter: fuel consumption/ pollutant to energy provided

$\beta_2$ = efficiency parameter: fuel consumption/ pollutant during positive acceleration to energy time acceleration

$\Delta F$ = fuel consumed or pollutant emitted during time  $\Delta t$

$\Delta t$ = time period

For CO<sub>2</sub>:

$$\Delta F = f_{CO_2} \Delta F(\text{fuel})$$

Where:  $f_{CO_2}$ =CO<sub>2</sub> rate (g/mL)

$\Delta F(\text{fuel})$ =Fuel consumption in mL

Parameter	Fuel	CO	HC	NO <sub>x</sub>
$f_i$	1350mL/h (LV) 2000mL/h (HV)	50g/h	8g/h	2g/h
$10^4\beta_1$	900mL/kJ (LV) 800mL/kJ (HV)	150g/kJ	0g/kJ	10g/kJ
$10^4\beta_2$	300mL/(kJm/s <sup>2</sup> )(LV) 200mL/(kJm/s <sup>2</sup> )(HV)	250g/(kJm/s <sup>2</sup> )	4 g/(kJm/s <sup>2</sup> )	2 g/(kJm/s <sup>2</sup> )
$M_{vLV}$	1400 kg	1400kg	1400kg	1400kg
$M_{vHV}$	11000 kg	11000kg	11000kg	11000kg
CO <sub>2</sub> : $f_{CO_2LV}=2.5$ g/ml $f_{CO_2HV}=2.6$ g/ml				

Figure 2-3: Parameters used in SIDRA emissions model[16]

Unlike Grader, other authors' environmental results are varied from small reductions to increases in fuel consumption and emissions when using a roundabout over other alternatives, to large reductions in fuel consumption and emissions. One such study proposes that when comparing to an unsignalized intersection, the roundabout increases emissions by 4% in nitrogen oxides (NO<sub>x</sub>) and 6% in carbon dioxide (CO<sub>2</sub>) and when compared to pre-timed signals, they decrease NO<sub>x</sub> by 21% and CO<sub>2</sub> by 29% [17]. This is also suggested by Hoglund as his study explained that the uniform speeds around a roundabout would allow for a reduction from a stopped or signalized intersection where most cars must stop. [18].

Similar results were found when comparing specific roundabouts with stopped signals. In two different studies, Mandavilli et al. compared six intersections before and after a roundabout was constructed. They demonstrated there was a reduction in all four MOEs he used when the peak hour volume was between 192 and 1,220 veh/hr. In this case study, hydrocarbons were reduced by 17% to 65% of emissions at the original intersection. Carbon dioxide was reduced by 21% to 42%, NO<sub>x</sub> was reduced by a 20% to 48%, and CO<sub>2</sub> was reduced by 15% to 59%. [19 20]

Vlahos, Polus et al. found smaller improvements but just as impressive. Using SIDRA and demand data from a pair of roundabouts of 16,350 AADT and 14,000 AADT, They showed an overall

reduction in delay, queues and all emissions (see Figure 2-4) for both a 1.00 Volume to Capacity ratio (V/C) and a 1.30 V/C. They also showed that with different turning movements and heavy vehicles the emissions also decreased. These researchers go on to discuss the improvements of a roundabout over a signalized intersection. They present a data table (see Figure 2-5) of cross over points where a roundabout is no longer better than a signal. [21]

Descriptor	Ratios	Effective Intersection Capacity	Degree of Saturation	Queue Length	Average Delay	Emissions			
						CO <sub>2</sub>	HC	CO	NO <sub>x</sub>
Volume ratio	1.00	186.1	64.7	38.6	48.1	10.1	16.1	18.6	11.5
	1.33	166.8	61.4	34.2	46.8	10.0	16.4	19.1	11.7
Turning percentages	(10, 80, 10)	186.1	64.7	38.6	48.1	10.1	16.1	18.6	11.5
	(20, 70, 100)	215.6	68.0	49.0	50.9	10.4	16.5	17.6	10.9
	(30, 60, 10)	257.2	71.7	59.1	56.0	11.8	18.4	17.1	10.9
Heavy vehicle percentages	(10, 5)	187.0	64.8	38.1	48.1	10.1	16.1	18.6	11.2
	(10, 10)	186.1	64.7	38.6	48.1	10.1	16.1	18.6	11.5
	(20, 10)	165.4	61.9	37.1	48.2	10.2	16.6	18.5	11.9
	(20, 20)	165.3	61.8	36.8	48.1	9.9	16.8	18.3	12.4

Figure 2-4: Percent improvement using a roundabout over a All-way stop controlled intersection [21]

Descriptor	Turn Ratio (10, 80, 10)				Turn Ratio (20, 70, 10)				Turn Ratio (30, 60, 10)			
	(10,5)	(10,10)	(20,10)	(20,20)	(10,5)	(10,10)	(20,10)	(20,20)	(10,5)	(10,10)	(20,10)	(20,20)
A. For Volume Ratio 1.0												
V/C	2500	2400	2250	2150	2380	2350	2040	2100	2300	2250	2050	2020
Capacity	2500	2450	2300	2150	2380	2350	2170	2100	2300	2250	2050	2020
Queue	2700	-	2500	-	2580	-	2315	-	2470	-	-	-
Delay	2620	2550	2390	2230	2500	2400	2250	2140	2380	2300	2125	2040
CO <sub>2</sub>	2680	2600	2420	2290	2540	2460	2350	2190	2420	2350	2180	2100
HC	2660	2590	2420	2280	2540	2440	2300	2180	2420	2360	2190	2100
CO	2100	2100	1800	1500	2000	2000	1600	1000	1900	1800	1500	1300
NO <sub>x</sub>	2500	2500	2100	1900	2400	2300	1800	1100	2200	2100	1600	1500
B. For Volume Ratio 1.33												
V/C	2480	2800	2280	2200	2380	2320	2150	2050	2240	2170	1980	1960
Capacity	2480	2800	2280	2190	2380	2320	2160	2050	2260	2180	1980	1980
Queue	-	-	2450	-	-	2510	2300	-	2420	-	2140	2060
Delay	2620	2900	2380	2240	2470	2400	2220	2110	2320	2260	2100	1980
CO <sub>2</sub>	2800	2900	2400	2300	2540	2460	2380	2160	2380	2320	2140	2020
HC	2800	2980	2400	2350	2530	2460	2380	2160	2380	2320	2140	2020
CO	2100	2400	1900	1700	2010	1900	1920	1750	1920	1750	1575	1490
NO <sub>x</sub>	2500	2800	2200	1750	2400	2300	2000	1750	2180	1920	1750	1650

Figure 2-5: Maximum Demand for a roundabout when compared to a signal [21]

However, Ahn et al. showed that for a specific Origin and Destination (OD) demand and high speed on approaches the roundabout would actually cause more emissions than that of a stop or a signal. This OD has a very low side street demand as compared to the main street.[22] This is contrary to most design guides which suggest that a more even demand for all approaches would allow for better flow

around the roundabout. The FHWA suggests that the minor approaches should have 40% to 50% of the total demand for an intersection to have a viable roundabout.[2]

## 2.5 Conclusions

This review gave some insight as to how to calculate delay and capacity, what design factors may be important in distinguishing whether a roundabout will be more optimal than another intersection, and how other studies compared a roundabout. When comparing a roundabout many of these studies used a base case from a particular intersection and discussed how that could be generalized; some papers showed that these generalizations can be faulty. Although there has been much done in the realm of roundabouts much of the work has been in before and after studies with SIDRA as the main modeling tool. As also seen for delay analysis the research and leg work had been done already. In other design guides there is a failure to discuss the use of a roundabout over other alternatives. This thesis will recommend when roundabouts could be a better alternative to other intersection controls for uniform demands and 35mph approach speeds.

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## **Chapter 3: Do Roundabouts Work? An Evaluation for Uniform Approach Demands**

Meredith Jackson and Hesham Rakha, submitted to the TRB for 2012

### **3.1 Abstract**

With the increased prevalence of roundabouts in the United States, there is a need to evaluate the performance of roundabouts relative to other intersection control strategies. Few studies have compared roundabouts with other intersection control strategies in a systematic fashion. Consequently, this paper compares four types of intersection control strategies considering a single lane approach with a 35 mph speed limit and equal demand on all approaches. The study demonstrates that vehicle delay is minimized with the use of a roundabout intersection control for all demand levels below 500 veh/hr/approach. Above this point if the left turn percentage exceeds 70% traffic signal control is more efficient. The roundabout alternative also produces the fewest vehicle stops for low demand levels, low left turn demand and high right turn demand, however a TWSC alternative produces the least number of vehicle stops when the through and total demand is high. The study demonstrates that for low traffic demand levels roundabouts should be a part of design alternatives considered for isolated intersection control. The paper also shows that for over-saturated conditions a traffic signal does not necessarily produce the least delay by using the Webster-Cobb method; instead the shortest cycle length with the simplest phasing scheme may result in less vehicle delay and stops.

### **3.2 Introduction**

Today in the United States roundabouts are increasing in numbers.[1] Many communities across the US have seen a new roundabout put in place by their state department of transportation in conjunction with local municipalities to ease congestion or increase safety. Roundabouts have been a part of the traffic system in other forms since 1905 when the first circular intersection was constructed in New York City. This intersection gave entering vehicles right of way which facilitated high speed entries but the intersection, referred to as a rotary experienced high congestion and high accident rates [1]. After the 1950's rotaries and other forms of circular intersections fell out of style mostly because of the negative experience with circular intersections of the time [1]. In 1966 the first modern roundabout was developed in the United Kingdom [1]. Rather than priority given to entering vehicles circulating vehicles were given right of way and entry and exiting traffic was channelized increasing safety and efficiency of circulating traffic.

In the 2000 Highway Capacity Manual (HCM2000), roundabouts were given a subchapter to facilitate design guides; however a simple graph (Figure 3-1) is included to show the ranges of demands that three types of intersection control are considered optimum. This graph states that for certain ranges a roundabout can be better. However it does not discuss what the ranges are. For major and minor peak hour volumes there are ranges that two-way stop control (TWSC), all-way stop control (AWSC) and traffic signal control. Over the red line, two-way peak hour volume shows that for demands lower than 400 vehicles/hour (veh/hr) a TWSC is the optimum intersection control strategy, between 400 and 600 veh/hr AWSC is optimum and for demands greater than 600 veh/hr a signal is optimum (HCM 2000).

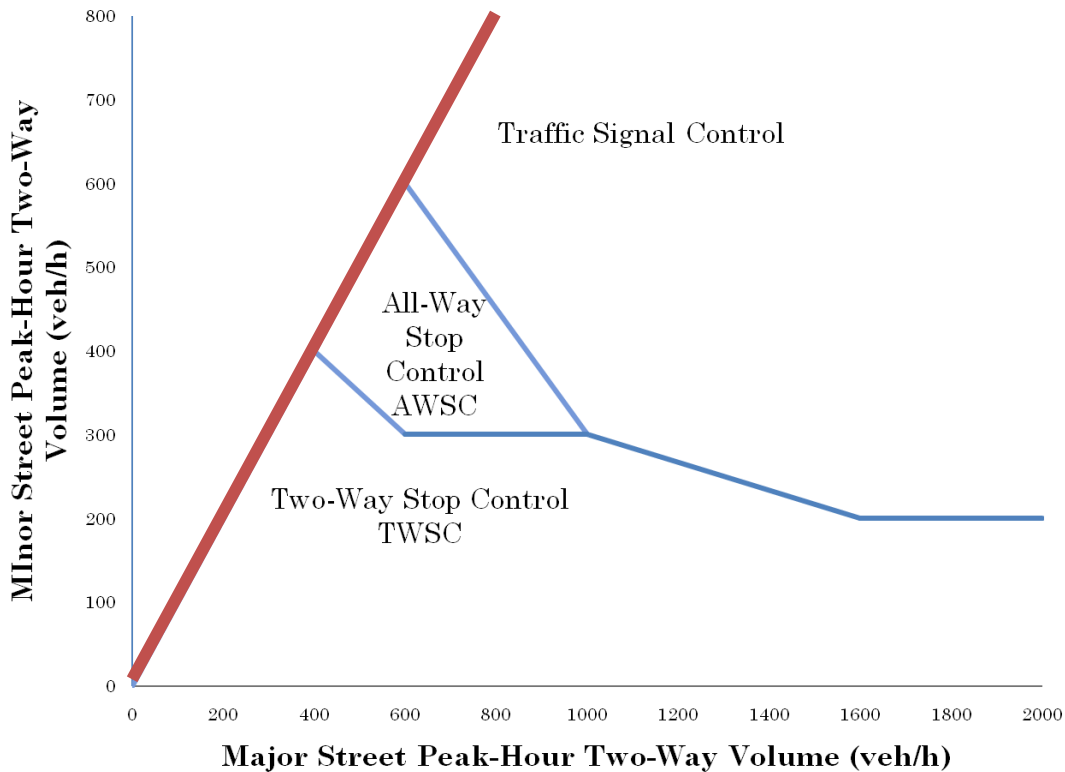


Figure 3-1 : Volumes Recommended for Different Intersection Control Strategies. Roundabouts may be a viable alternative for portions of these ranges. [2]

The objective of this paper is to identify where along the thick red line of Figure 3-1 roundabouts are more efficient with respect to other intersection control strategies considering two measures of effectiveness, namely vehicle delay and stops.

### 3.3 Literature Review

With the availability of different types of intersection control, designers must decide which would be best to use based upon traffic demands, geography, and geometry. Using some dynamic modeling, Zhang et al. compared a pre-timed signal control to a roundabout and showed that for longer queues the signal will have lower delays while for shorter queues a roundabout will have shorter delays [3].

In another study simulating two TWSC, two AWSC, and one roundabout intersection using the SIDRA software, Russell et al. showed that a roundabout when used in the proper situation, with even approach demands, speeds less than 25 mph (40 kph), and proper geometric design, performed better in most standard MOEs when compared to TWSC and AWSC intersection control. They found that a TWSC and a roundabout have essentially the same or similar average delay.[4]

Sisioupiku and Oh [5] compared TWSC, AWSC, signal control, and yield control to a Roundabout. They used volume ratio approaches that were even for all approaches and left turning percentages of 10%, 20% and 30% and assumed the defaults of the SIDRA program for the roundabout geometry. In their comparison they showed that for light demands, TWSC and yield control can be highly effective. For most demands they found that AWSC has higher delay than either roundabouts or signalized intersections. For one-lane approaches, signals and

roundabouts are comparable alternatives, for two-lane approaches with heavy volumes; roundabouts have lower delays than signals and for three-lane approaches, signal controls are the more viable option. They also showed that for heavy left turns two lane roundabouts have lower delay and increased capacity. [5]

Nevada Department of Transportation redesigned a four-way single-lane approach intersection with peak hour volumes between 160 and 740 vehicles per hour. The NETSIM software was used to simulate an AWSC, a signal control, and a roundabout. The study showed that with these conditions the roundabout will decrease the average travel time for the entire 30 minutes of simulation. [6]

Before and after studies done in Kansas and other areas around the US and through Europe often compared the traditional intersection that was replaced with a newer roundabout. These studies used demands found on-site about the time of the change and SIDRA to model the MOEs. In most of these studies the researchers found that roundabouts were comparable if not better than the previous intersection control method. In a Kansas study the comparison (though qualitative) was clear and shows that for the cases compared the roundabout was a viable option [7]. This was also shown in quantitative analysis around three different roundabouts in New Hampshire, New York and Washington. These roundabouts had a daily volume of 7,600-11,250 vehicles. They showed an 83 to 93% decrease in average intersection delay and a 31 to 87% reduction in total stops. [8]

### **3.4 Study Methodology**

Several examples of comparing roundabouts and other intersection control have been conducted, however none of these can be really generalized and many have conflicting views as to whether signals or roundabouts are “better” for different traffic demand configurations. To start that general comparison, a simplified intersection should be considered. The intersection considered has four approaches each with only a single lane and no pocket lanes. Uniform Flows between 200 veh/hr/ln and 1600 veh/hr/ln in increments of 200 veh/hr/ln and turn percentages between 0% and 100% in increments of 10% were used for the demands at the intersections. Four controls were considered: TWSC, AWSC, roundabout and traffic signal. The design speed for the intersection was 35 mph or 58 km/hr.

In earlier publications, SIDRA, VISSIM or NETSIM were used to conduct the analysis, however in this paper the INTEGRATION software is used because the software has been validated against field data. An earlier study compared INTEGRATION and VISSIM results to field data for a basic intersection. The study concluded that the INTEGRATION output was more consistent with the queue length field observations at a TWSC intersection. The study showed that VISSIM queue lengths were slightly longer than the field observations and that average travel time was comparable for both models. [9].

The INTEGRATION simulation runs were made for the peak 15 minutes for each scenario considering a uniform demand. Approaches set to be 3 km in length to ensure that any queues would not spill back beyond the simulation geometric boundaries. A saturation flow rate of 1800 veh/hr was assumed for all intersection alternatives. All simulations were executed to clear all vehicles that entered the network.

The design of the roundabout has to be considered. Based on the literature [1] the maximum suggested speed for circulating traffic in a single lane roundabout should be no more than 25 mph (40 km/hr). This design speed is used for the circulating traffic in the roundabout

so that the speed variation between the free-flow speed on the approach roads is minimal. This design speed corresponds to a roundabout with a 40 m diameter [1].

### 3.5 Traffic Signal Timing Estimation

Traffic signal timings were initially computed using the Webster-Cobb Method as discussed in HCM 2000. A two-phase scheme was first considered followed by a four phase scheme in which each approach had its own phase. No protected left turn phases were considered given that the signalized approaches were single shared lanes. For under-saturated conditions, a two phase configuration was simulated with cycle lengths ranging between 30s and 180s. For these conditions simulation results and analytical estimates of delay were similar and thus the Webster-Cobb method was used. However in over-saturated conditions the simulation delay was much higher than expected. Therefore experimentation was needed to see if other timing lengths and phase configurations would produce shorter delay estimates. Two other signal timing schemes were simulated for over-saturated conditions: one with four phases operating at the minimum cycle length of 50s and the other with two phases at the minimum cycle length of 30s.

Figure 3-2 displays the timing plan that produces the lowest overall vehicle delay. Plan 1 or the four-phase plan operating at the maximum cycle length (180s cycle length) the two-phase plan at the minimum cycle length (30s cycle length) or plan 2 and plan 3, the four-phase plan operating at its minimum cycle length (50s cycle length) are represented by red, yellow and green shades, respectively. For oversaturated conditions Figure 3-2 shows that plan 2 results in the minimum intersection delay. This result seems to suggest that sneakers utilize every yellow and all-red period at the end of green and with more inter-green time more left turning vehicles could sneak lowering the delay. On average plan 2 was found to be 2.63 times better than plan 1 in oversaturated conditions and 2.35 times better than plan 3.

Figure 3-3 demonstrates the overall trend that a two-phase operation at the minimum cycle length produces the least vehicle delay and therefore should be used for oversaturated conditions. In order to compare the delay estimates associated with the three plans a more detailed analysis was conducted. In Figure 3-3 the effects of increasing demand and left or right turn percentages on the overall intersection delay are illustrated. Plan 1, 2, and 3 are displayed in green, red and orange lines, respectively. In all four graphs the y axis is the average delay measured in s/veh.

Four notable basic trends are illustrated in Figure 3-3. As expected, while maintaining a constant left and right turn demand of 10% and 30%, respectively the delay increases for all three plans as demand increases. The average delay for plan 2 increases from 21 s/veh at an approach demand of 200 veh/hr to 93.27 s/veh at a demand of 800 veh/hr/approach and 488.45 s/veh at 1600 veh/hr/approach. This increase in delay is significantly lower than in the case of plan 1 and 3. Specifically, the delay increases by 3.6 and 3.8 times for plan 1 and plan 3 for the 800 veh/hr/approach demand and by 2.2 times for plan 1 and plan 3 for a 1600 veh/hr/approach demand.

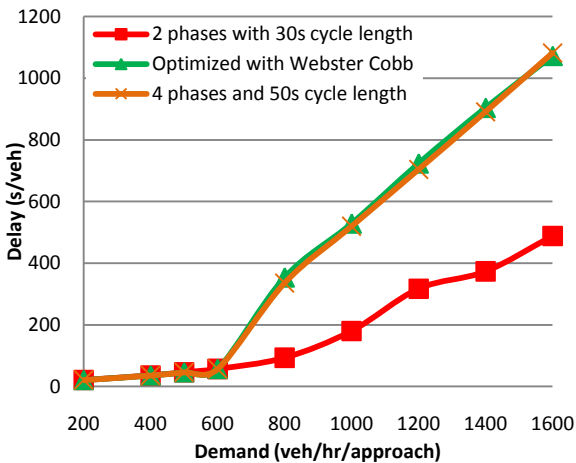
Unlike the total demand trend, right turn percentages has very marginal to no effect on the average vehicle delay. Plan 2 has a marginal increase in delay with increase in right turn percentages. In this example shown in Figure 3-3 when demand is a constant 1400 veh/hr/approach and left turns are a constant 30%, delay for plan 2 increases from 439.38 s/veh when there is no right turn percentage to 496.72 s/veh when there is 70% right turn percentage. However, delay for both plan 1 and 3 is unaffected by right turn percentage. The delay for plan 3 averages about 877 s/veh and plan 1 averages 898.6s/veh.

Like right turn percentages effects on delay for demand at saturated conditions, left turn percentage has very marginal effects on delay when left turn percentages are between 20% and 80%. The example keeps demand constant at 500 veh/hr/approach and right turn percentage constant at 10%. In fact there is more noise than increases or decreases. Delay for plan 2 is 43.2 s/veh at 30% left turns and 43.75 s/veh for 70% left turns. Delay for plan 3 averages 2 times greater than the delay for plan 2 and delay for plan 1 averages 3.1 times the delay for plan 2.

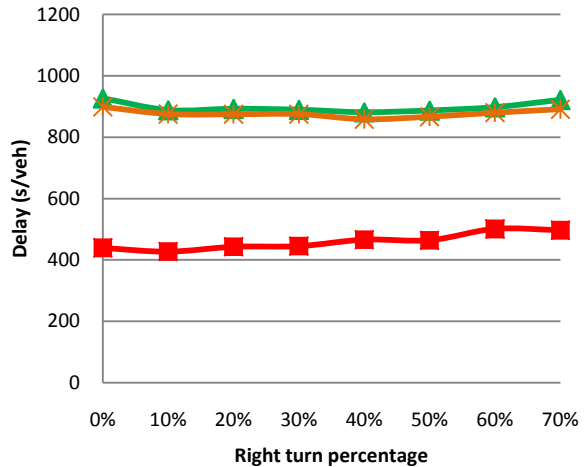
For higher demand levels (e.g. 1600 veh/hr/approach), delay for plan 2 increases from 449 s/veh for no left turns to 495.93 s/veh at 10% left turns to 551.52 s/veh at 20% left turns and decreases to 386.17 s/veh for 90% left turners. The opposite is true for both plan 1 and plan 3. Delay for plan 3 decreases from 1188.5 s/veh at 10% left turns to 1045.4 s/veh at 50% and then increases to 1030.1 s/veh at 90% left turns. Delay for plan 1 decreases from 1148.5s/veh at 10% left turns to 1071.3 s/veh at 50% left turns and also increases to 1091.9 s/veh at 90% left turns.

Left	right	200	400	500	600	800	1000	1200	1400	1600	
0%	0%	1	1	1	1	1	2	2	2	2	
	10%	1	1	1	1	1	2	2	2	2	1 HCM
	20%	1	1	1	1	1	2	2	2	2	2 2phase, min cycle
	30%	1	1	1	1	1	2	2	2	2	3 4phase min cycle
	40%	1	1	1	1	1	2	2	2	2	
	50%	1	1	1	1	1	2	2	2	2	
	60%	1	1	1	1	1	2	2	2	2	
	70%	1	1	1	1	1	2	2	2	2	
	80%	1	1	1	1	1	2	2	2	2	
	90%	1	1	1	1	1	2	2	2	2	
	100%	1	1	1	1	1	2	2	2	2	
10%	0%	1	1	1	1	2	2	2	2	2	
	10%	1	1	1	1	2	2	2	2	2	
	20%	1	1	1	1	2	2	2	2	2	
	30%	1	1	1	1	2	2	2	2	2	
	40%	1	1	1	1	2	2	2	2	2	
	50%	1	1	1	1	2	2	2	2	2	
	60%	1	1	1	1	2	2	2	2	2	
	70%	1	1	1	1	2	2	2	2	2	
	80%	1	1	1	1	2	2	2	2	2	
	90%	1	1	1	1	2	2	2	2	2	
20%	0%	1	1	1	2	2	2	2	2	2	
	10%	1	1	1	2	2	2	2	2	2	
	20%	1	1	1	2	2	2	2	2	2	
	30%	1	1	1	2	2	2	2	2	2	
	40%	1	1	1	2	2	2	2	2	2	
	50%	1	1	1	2	2	2	2	2	2	
	60%	1	1	1	2	2	2	2	2	2	
	70%	1	1	1	2	2	2	2	2	2	
	80%	1	1	1	2	2	2	2	2	2	
30%	0%	1	1	2	2	2	2	2	2	2	
	10%	1	1	2	2	2	2	2	2	2	
	20%	1	1	2	2	2	2	2	2	2	
	30%	1	1	2	2	2	2	2	2	2	
	40%	1	1	2	2	2	2	2	2	2	
	50%	1	1	2	2	2	2	2	2	2	
	60%	1	1	2	2	2	2	2	2	2	
	70%	1	1	2	2	2	2	2	2	2	
40%	0%	1	1	2	2	2	2	2	2	2	
	10%	1	1	2	2	2	2	2	2	2	
	20%	1	1	2	2	2	2	2	2	2	
	30%	1	1	2	2	2	2	2	2	2	
	40%	1	1	2	2	2	2	2	2	2	
	50%	1	1	2	2	2	2	2	2	2	
	60%	1	1	2	2	2	2	2	2	2	
50%	0%	1	1	2	2	2	2	2	2	2	
	10%	1	1	2	2	2	2	2	2	2	
	20%	1	1	2	2	2	2	2	2	2	
	30%	1	1	2	2	2	2	2	2	2	
	40%	1	1	2	2	2	2	2	2	2	
	50%	1	1	2	2	2	2	2	2	2	
60%	0%	1	1	2	2	2	2	2	2	2	
	10%	1	1	2	2	2	2	2	2	2	
	20%	1	1	2	2	2	2	2	2	2	
	30%	1	1	2	2	2	2	2	2	2	
	40%	1	1	2	2	2	2	2	2	2	
70%	0%	1	1	2	2	2	2	2	2	2	
	10%	1	1	2	2	2	2	2	2	2	
	20%	1	1	2	2	2	2	2	2	2	
	30%	1	1	2	2	2	2	2	2	2	
80%	0%	1	1	1	2	2	2	2	2	2	
	10%	1	1	1	2	2	2	2	2	2	
	20%	1	1	1	2	2	2	2	2	2	
90%	0%	1	1	1	1	2	2	2	2	2	
	10%	1	1	1	1	2	2	2	2	2	
100%	0%	2	2	1	2	2	2	2	2	2	

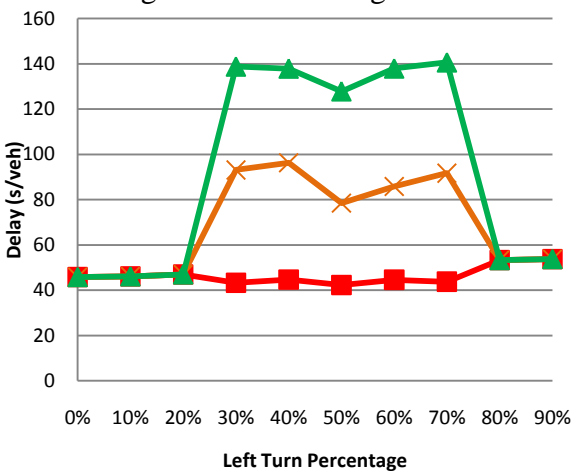
Figure 3-2: Phase Scheme Producing Minimum Delay as a Function of Demand Level and Configuration.



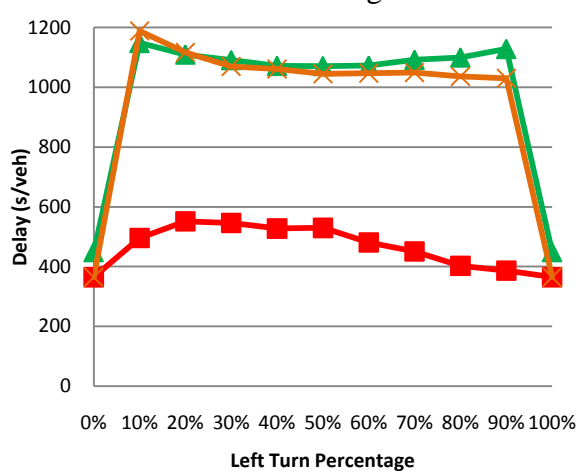
(a) Demand vs. Delay  
Left turn percentage: 10%  
Right Turn Percentage: 30%



(b) Right Turn Percentage vs. Delay  
Demand: 1400 veh/hr/approach  
Left Turn Percentage: 30%



(c) Left Turn Percentage vs. Delay  
Demand: 500 veh/hr/approach  
Right Turn Percentage: 10%



(f) Left Turn Percentage vs. Delay  
Demand: 1600 veh/hr/approach  
Right Turn Percentage: 0%

Figure 3-3: Comparing Signal Timings for Over-saturated Conditions based on Average Delay. Demands are in veh/hr/approach, Delay is in s/veh

In contrast to the least delay plan for over-saturated conditions, the average number of stops is not always minimized in plan 2. As seen in Figure 3-4, plan 1, plan 2 and plan 3 are represented by red, yellow and green, respectively. The average number of stops is minimized with plan 3 when overall demand is between 1000 veh/hr/approach and 1400 veh/hr/approach and through traffic demand is small compared to left and right turn demand.

Underlying trends are exemplified in Figure 3-5. The signal with plan 1, plan 2 and plan 3 are represented by a green line, a red line and orange line respectively. In all four graphs the y axis is the average number of stops.

In Figure 3-5, some of the trends are very similar to the delay trends that were presented earlier. However, with increasing demand, a very different pattern emerges. In the example where left turns are a constant 10% and Right turns are a constant 90%, the number of vehicle

stops for all three signal timing plans increase and then start to level off with increasing demands. For all designs, when demand is 600 veh/hr/approach, the average number of stops is 0.881 stops/veh. After the conditions become over saturated, both plan 2 and plan 3 increase similarly. When demand is 1000 veh/hr/approach, the number of stops for plan 3 is 1.923 stops/veh and is less than the number of stops associated with plan 2 which is 1.956 stops/veh. When demand is 1200 veh/hr/approach the number of stops increase to 2.2 stops/veh and 2.098 stops/veh for plans 3 and plan 2, respectively. The number of stops increases to 2.373 stops/veh and 2.608 stop/veh when demand increases to 1600 veh/hr/approach. The number of vehicle stops for plan 1 increases significantly to 2.932 stops/veh or 1.5 times the number of stops for plan 2 at 1000 veh/hr/approach then increases to 3.336 stops/veh or 1.6 times the number of stops for plan 2 when the demand is 1200 veh/hr/approach and 3.812 stops/veh or 1.6 times the number of stops of plan 2 when the traffic demand is 1600veh/hr/approach.

When stops are studied in relation to right turn percentages, the trends are similar to that of delay with one difference. As exemplified when holding the demand constant at 1400 veh/hr/approach and the left turn percentage at 10%, the number of stops for plan 2 marginally increases from 2.221 stops/veh for no right turns to 2.322 for 90% right turns. Unlike delay, the number of vehicle stops for plan 3 is only about 1.1 times the number of vehicle stops for plan 2. For plan 1 and plan 3 there is no correlation between increasing right turning demand and the number of vehicle stops. For plan 3, the number of vehicle stops is 2.436 stops/veh for no right turns and 2.438 stops for 90% right turning traffic. For plan 1 when there are no right turns there are 3.637 stop/veh and at 90% right turns there are 3.628 stops/veh. These numbers average somewhere between 2.07 and 2.13 times the number of stops for plan 2.

Like right turn percentage effects on the number of vehicle stops, for demand at saturated conditions, left turn percentage has very marginal effects on the number of vehicle stops when the left demand is between 20% and 80% of the total demand. In this example the demand is constant at 600 veh/hr/approach and right turn percentage is constant at 10%. In fact there is more noise than increases or decreases in the number of vehicle stops. The number of vehicle stops for plan 2 is 0.831 stops/veh at 20% left turns and 0.764 stop/veh for 80% left turns. Vehicle stops for plan 1 average twice as great as the number of vehicle stops for plan 2 and the average number of vehicle stops for plan 3 averages 1.57 times the number of vehicle stops for plan 2.

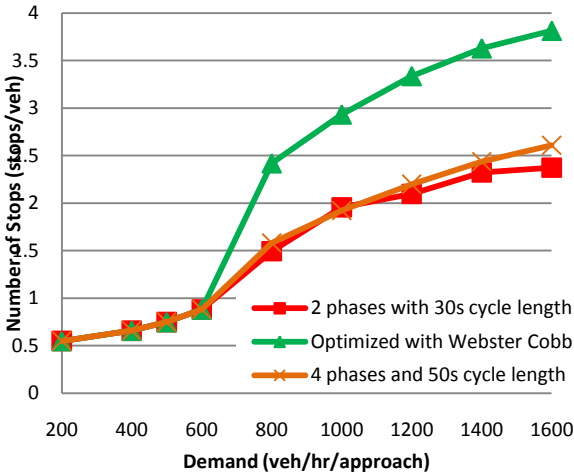
When conditions are highly over saturated like when the approach demand is 1600 veh/hr/approach with no right turns, left turns affect the number of stops to a greater degree. For plan 2, stops increase from 2.251 stops/veh at 10% left turns to 2.408 stops/veh at 20% left turns. From 20% to 100% left turn the number of stops decreases to 2.036 stops/veh at 100% left turns. In contrast plan 3 and plan 1 marginally decrease the number of stops from 2.622 and 3.841 stops/veh, respectively when the left turn percentage is 10% to 2.566 and 3.717 stops/veh, respectively when the left turn percentage is 50% and marginally increases to 2.6 and 3.708 stops/veh when left turn percentage is 90%.

In summary, the number of vehicle stops and delay are not minimized for the same signal phasing scheme for all demand levels and turning percentages. This could cause some difficulties when it comes to comparing signals with other control strategies. However, current standards have mostly attempted to minimize delay and not other measures of effectiveness. Given this logic, in oversaturated conditions, the signal plans used in the comparison against roundabouts and stop control alternatives is the signal with two phase and 30s cycle length because it has the least delay of all plans.

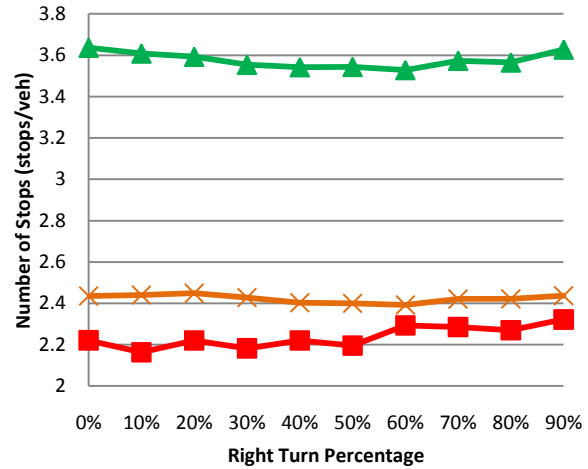


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	10%	1	1	1	1	1	3	3	3	3	1 HCM
	20%	1	1	1	1	1	3	3	3	3	2 2phase,min cycle
	30%	1	1	1	1	1	3	3	3	3	3 4phase min cycle
	40%	1	1	1	1	1	3	3	3	3	
	50%	1	1	1	1	1	3	3	3	3	
	60%	1	1	1	1	1	3	3	3	3	
	70%	1	1	1	1	1	3	3	3	3	
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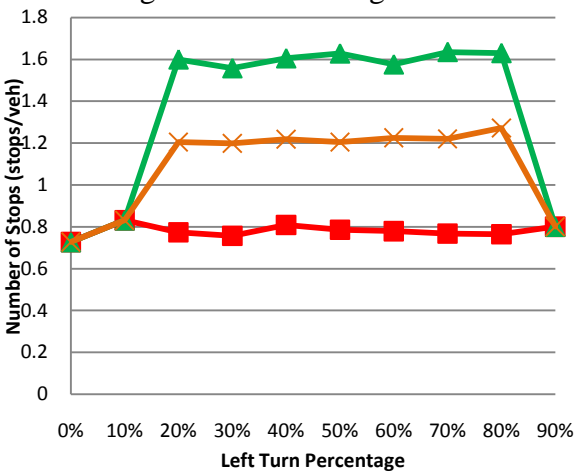
Figure 3-4: Phase Scheme Producing Minimum Number of Vehicle Stops as a Function of Total Demand and Demand Distribution



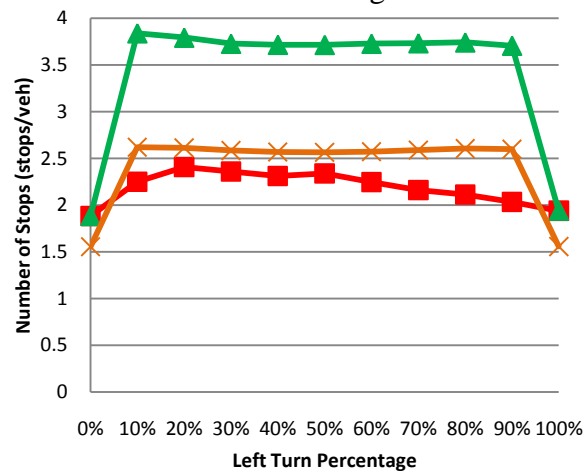
(a) Demand vs. Number of Stops  
Left turn percentage: 10%  
Right Turn Percentage: 90%



(b) Right Turn Percentage vs. Number of Stops  
Demand: 1400 veh/hr/approach  
Left Turn Percentage: 10%



(c) Left Turn Percentage vs. Number of Stops  
Demand: 600 veh/hr/approach  
Right Turn Percentage: 10%



(d) Left Turn Percentage vs. Number of Stops  
Demand: 1600 veh/hr/approach  
Right Turn Percentage: 0%

Figure 3-5: Number of Vehicle Stop Variation for Signal Timings under Over-saturated Conditions

### 3.6 Comparison of Intersection Control Strategies Based On Vehicle Delay

Using the signal timing plans that minimize delay, a signal intersection control alternative was compared to an AWSC, a TWSC and a roundabout alternative. As seen in Figure 3-6 and Figure 3-7, roundabouts and traffic signals provide good control strategies to minimize vehicle delay. In Figure 3-6, the traffic signal, roundabout, AWSC and TWSC alternative are represented by green, yellow, orange, and red colors, respectively. Figure 3-6 demonstrates that the roundabout alternative has the least delay of the four intersection control strategies for left turn percentages below 40% of all demand, when the overall demand is less than 500 veh/hr/approach or when left turn percentage is between 40 and 70% and through traffic demand is greater than 600 veh/hr/approach. Otherwise the signal alternative has the least delay.

Figure 3-7 exemplifies underlying trends in the data. The traffic signal, TWSC, AWSC and roundabout alternatives are represented by the colors red, orange, gold, and green, respectively. The y-axis is the average delay in seconds/vehicle (s/veh).

When comparing total demand versus delay, at any turn percentage when demand increases the average delay increases. For example when the left turn demand is 10% and right turn demand is 40% and the total demand is low there is very little difference in delay between the four intersection control strategies. Specifically, at an approach demand of 200 veh/hr/approach, delays for the signal, TWSC, AWSC and roundabout alternatives are 21.40 s/veh, 21.42 s/veh, 27.59 s/veh and 17.10 s/veh, respectively. Up to approach demands of 600 veh/hr/approach, all four intersection control strategies can be considered to have comparable delays. At 600 veh/hr/approach delays for the TWSC, AWSC, roundabout and signal alternatives are 74.886 s/veh, 61.48 s/veh, 42.332 s/veh and 59.026 s/veh, respectively. As the total demand increases, the difference in delay associated with the four intersection control strategies increases. At 1600 veh/hr/approach the TWSC alternative has the greatest delay of 830.75 s/veh and the roundabout alternative has the least delay of 282.27 s/veh. For the AWSC and signal alternative the delays are 676.44 s/veh and 516.01s/veh, respectively. When the left turn percentage is high and right turn percentage is low as in the example of 80% left turns and 10% right turning vehicles, when the total demand level is less than 400 veh/hr/approach, the result is similar to the previous example. Delay for an approach demand of 400 veh/hr/approach for the TWSC, AWSC, roundabout and traffic signal alternatives are 34.45 s/veh, 41.244 s/veh, 30.339 s/veh and 38.285 s/veh, respectively. However, for demands greater than 400 veh/hr/approach, the traffic signal control alternative produces the least delay. At 600 veh/hr/approach the delays for the TWSC, AWSC, roundabout and traffic signal alternatives are 113.13s/veh, 97.72 s/veh, 56.361 s/veh, and 52.173 s/veh, respectively. At 1600 veh/hr/approach the signal alternative now has the least delay of 436.86 s/veh and the TWSC and AWSC alternatives have similar delays of 784.86 s/veh and 783.04 s/veh, respectively. At 1600 veh/hr/approach for the roundabout alternative produces an average vehicle delay of 674.05 s/veh.

As seen in Figure 3-7, when comparing right turn percentage vs. delay, at low demand levels there are marginal effects of increasing right turn percentages. In the example of 500 veh/hr/approach and no left turning traffic, the delays for the TWSC, AWSC and roundabout alternatives marginally decrease from 43.17s/veh, 116.46 s/veh, 36.079 s/veh respectively when there are no right turns to 39.01 s/veh, 45.39 s/veh and 33.96 s.veh, respectively when all vehicles are turning right. For the traffic signal alternative, the average vehicle delay decreases marginally from 48.21 s/veh when there are no right turning vehicles to 43.55 s/veh when half the vehicles are right turners and then marginally increases to 48.21 s/veh when all the vehicles are right turners. For greater demands as in the example of 1600 veh/hr/approach and no left turning vehicles the trends change. At both the signal and AWSC right turning traffic has no discernable impact on delay. When there is no right turning traffic the AWSC and the traffic signal alternatives produce average delays of 592.17 s/veh and 364.52 s/veh, respectively. When all traffic is turning right the average delay is 590.38 s/veh and 364.52 s/veh, respectively. Delays for the TWSC and roundabouts alternatives decrease from 726.97 s/veh and 364.52 s/veh when there is no right turning traffic to 410.09 s/veh and 165.13 s/veh when all vehicles are right turners. Because of the decrease in delay for the roundabout and TWSC alternatives, when there is no right turning traffic a signal has the least delay and when 10% of the traffic is turning roundabouts and signal alternatives are comparable with a delay of 361.87 s/veh and 374.09

s/veh respectively, and with increasing right turn traffic after this point delay is minimized with the roundabout alternative. When half of the traffic demand turns right, the AWSC and the TWSC alternatives are comparable, after this point when increasing right turning traffic delay at the TWSC is less than the AWSC.

Similar to the marginal impact of right turning traffic on vehicle delay for low demand levels, delay is nearly unaffected by the left turn percentage when the approach demand is 200 veh/hr/approach and there is no right turning traffic. Delays for the TWSC, AWSC, and traffic signal alternatives average 20.79 s/veh, 26.98 s/veh, and 21.60 s/veh, respectively for all turning demands. Delay for the Roundabout alternative in this example marginally increases from 15.959 s/veh when there is no left turning traffic to 17.142 s/veh when all traffic is turning left. This differs from when the approach demand is higher (e.g. approach demand is 1400veh/hr/approach) and 20% of traffic is turning right. In this case, delay for the TWSC alternative increases from 563.8 s/veh when there are no left turning traffic to 718.3 s/veh when 10% of traffic is turning left then decreases to 671.88 s/veh when 80% of traffic is turning left. Similar to the TWSC alternative, delay for the AWSC alternative increases from 479.9 s/veh when no traffic turns left to 679.09 s/veh when 50% of traffic is turning left and decreases to 635.24 s/veh when 80% of traffic is turning left. In similar fashion delay for the signal alternative increases from 281.1 s/veh when there is no left turning traffic to 443.1 s/veh when 30% of traffic is turning left and decreases to 383.6 s/veh when 80% of traffic is turning left. Unlike the other three intersection alternatives, delay for the roundabout alternative steadily increases from 246.56 s/veh when there is no left turning traffic to 501.7 s/veh when 80% of vehicles are turning left.

### **3.7 Comparison Based On the Number Of Vehicle Stops**

Unlike the delay measure of effectiveness, the roundabout and TWSC alternatives typically minimize the number of vehicle stops. Figure 3-8 demonstrates how often the TWSC alternative has the fewest number of stops than other controls especially when left and through turning percentages are high and there is high demand. Conversely, when the right turning percentage is high and demand is low the roundabout alternative produces fewer vehicle stops. However, there are certain demands and turn percentages when the AWSC and traffic signal control strategies produce fewer vehicle stops.

Figure 3-9 illustrates examples of underlying trends in the data. The traffic signal, TWSC, AWSC, and roundabout alternatives are represented by the colors red, orange, gold and green, respectively. The y-axis is the average number of vehicle stops in stops/vehicle (stops/veh).

In Figure 3-9, six trends are exemplified. The first two examples are of trends in demand versus number of vehicle stops one with a high right turn percentage and one with a high left turn percentage. In the high right turn percentage case when 90% of traffic turns right while 10% turns left, the roundabout alternative has the fewest stops per vehicle at all demands. The number of stops for the roundabout alternative increases from 0.338 stops/veh when demand is 200 veh/hr/approach to 0.654 stops/veh when demand is 1600 veh/hr/approach with the greatest increase between 1000 veh/hr/approach and 1600 veh/hr/approach. For the signal alternative, the increase in the number of stops is much greater starting at 0.550 stops/veh when demand is 200 veh/hr/approach and reaching 2.37stops/veh when demand is 1600 veh/hr with the greatest rate of increase between 600 veh/hr/approach and 1000 veh/hr/approach. The number of vehicle stops for the TWSC alternative starts at 0.518 stops/veh and increases to 1.162 stops/veh with the

greatest increase between 600 veh/hr/approach and 1400 veh/hr/approach. The number of vehicle stops at the AWSC alternative have only marginal increases and decreases in this case and overall average about 1 stop/veh. In the opposite case in which 80% of vehicles turn left and 10% turn right stops at all intersection alternatives increase greatly. For the TWSC alternative, the number of vehicle stops increases from 0.529 stops/veh to 0.928 stops/veh between 200 veh/hr/approach and 800 veh/hr/approach with the greatest increase between 400 veh/hr/approach and 500 veh/hr/approach and then decreases to 0.861 stops/veh when the demand is 1600 veh/hr/approach. The number of stops at the AWSC alternative increases from 0.989 stops/veh to 1.382 stops/veh between 200 veh/hr/approach and 1000 veh/hr/approach and then decreases to 1.307 stops/veh at 1600 veh/hr/approach. For the roundabout alternative, the number of stops increases from 0.481 stops/veh when demand is 200 veh/hr/approach to 2.188 stops/veh when demand is 1600 veh/hr/approach with the greatest increase between 400 veh/hr/approach and 1000 veh/hr/approach. The number of vehicle stops for the traffic signal alternative increases from 0.555 stops/veh when demand is 200 veh/hr/approach to 2.184 stop/veh when demand is 1600 veh/hr/approach with the greatest increase between 600 veh/hr/approach and 1200 veh/hr/approach.

Unlike delay, right turn percentages do not affect the number of stops for the TWSC, AWSC, or traffic signal alternative. The average number of stops per vehicle for a demand of 200 veh/hr with 20% of traffic turning left are 0.540 stops/veh, 0.986 stops/veh, and 0.550 stops/veh, respectively. Like delay the number of stops for the roundabout alternative marginally decreases from 0.433 stops/veh to 0.359 stops/veh. Increasing the demand to 1400 veh/hr/approach with no left turns yields similar results for the AWSC and signal alternative. The average vehicle stops for these intersection control strategies are 1.050 stops/veh, and 1.917 stops/veh, respectively. For the TWSC alternative the number of vehicle stops increases from 0.761 stops/veh with no right turners to 0.835 stops/veh when half of the demand turns right and decreases marginally to 0.806 stops/veh when 70% of the demand turns right and finally more quickly decreases to 0.628 stops/veh when all vehicles are turn right. For the roundabout alternative the number of stops increases from 1.306 stops/veh with no right turners to 1.408 stops/veh when 10% of traffic is turns right and decreases to 0.295 stops/veh when all traffic is turns right, since there are no conflicting vehicles to worry about.

Left turns have more of an effect on stops than they have on delay when demand is low - in this case 400 veh/hr/approach with no right turning vehicles. The number of vehicle stops for the roundabout alternative increases with increasing left turning vehicles from 0.473 stops/veh when there are no left turning vehicles to 0.690 stops/veh when all vehicles turn left. For the AWSC alternative, the number of vehicle stops increases from 1.042 stops/veh when no traffic is turning to 1.134 stops/veh when 80% of traffic turns left, and then decreases to 1.080 stops/veh when all traffic is turn left. For the TWSC alternative, the number of vehicle stops increases from 0.538 stops/veh when no traffic turns left to 0.654 stops/veh when 40% of traffic is turns left and then decreases to 0.546 when all traffic turns left. The number of vehicle stops for the signal alternative increases from 0.615 stops/veh when no traffic turns left to 0.690 stops/veh when 30% of the demand turns left and decreases to 0.606 when all traffic is turns left.

### **3.8 Conclusions, Recommendations, and Further Research**

In this study two traditional MOEs were used to compare four alternative intersection control strategies (AWSC, TWSC, roundabouts, and traffic signals), namely: vehicle delay and stops. These comparisons were made considering a single lane approach with a 58 km/hr speed limit

and equal demand on all approaches. Delay, the measure typically used in the literature, is minimized with the use of a roundabout intersection control for all demand levels below 500 veh/hr/approach. Above this point if the left turn percentage exceeds 70% the traffic signal control becomes more efficient. The roundabout alternative also produces the fewest vehicle stops for low demand levels, low left turn demands, and high right turn demands; however TWSC produces the least number of vehicle stops when the through demand and total demand is high. The results demonstrate that for low demand levels roundabouts should be a part of design alternatives considered for isolated intersection control. In all conditions, the intersection that can be considered optimal is very different depending upon the turn percentages for both left and right turners as well as the geometry and overall demand. Also which measure of effectiveness to consider in the analysis may change the optimal intersection control for a given condition or set of conditions.

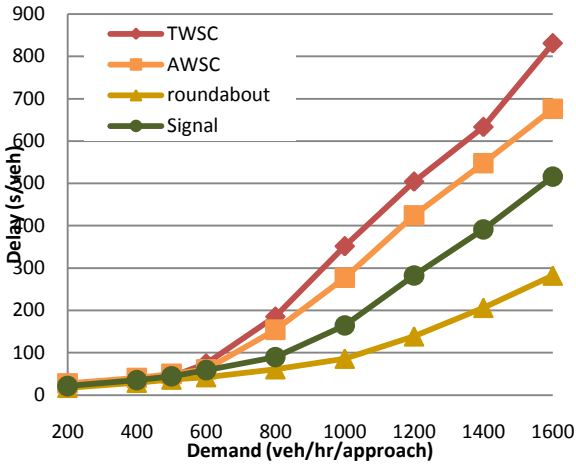
More research is needed to consider other factors. More demands with more independence between approaches is necessary to characterize the full impact a roundabout can have in an area especially when the demand is not equal on all approaches. This research dealt with a very specific geometry and speed limit. Consequently, the analysis should be extended to consider different geometries including more lanes and different sizes of roundabouts and speeds in order to generalize these findings to a larger domain of traffic conditions.

### 3.9 References

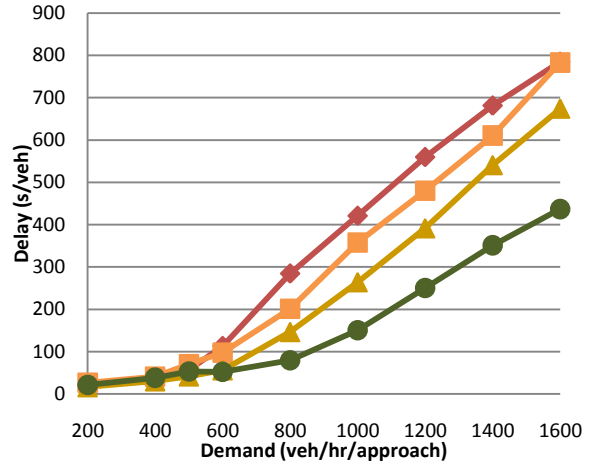
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Left	right	200	400	500	600	800	1000	1200	1400	1600	
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	10%	3	3	3	3	4	4	4	4	4	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
60%	0%	3	3	3	4	4	4	4	4	4	
	10%	3	3	3	3	4	4	4	4	4	
	20%	3	3	3	3	4	4	3	4	4	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
70%	0%	3	3	3	4	4	4	4	4	4	
	10%	3	3	3	3	4	4	4	4	4	
	20%	3	3	3	4	4	4	4	4	4	
	30%	3	3	3	3	4	3	4	3	4	
80%	0%	3	3	3	4	4	4	4	4	4	
	10%	3	3	3	4	4	4	4	4	4	
	20%	3	3	3	4	4	4	4	4	4	
90%	0%	3	3	3	3	4	4	4	4	4	
	10%	3	3	3	3	4	4	4	4	4	
100%	0%	3	3	3	4	4	4	4	4	4	

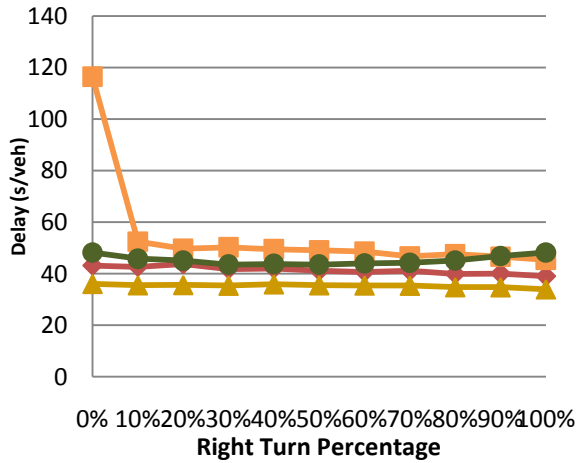
Figure 3-6: Control with Minimum Delay Grouped as a Function of Demand Level and Configuration. Demands are veh/hr/approach



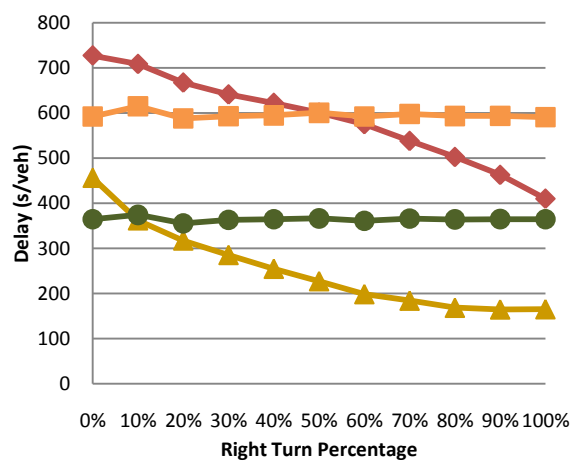
(a) Demand vs. Delay  
Left Turn Percentage: 10%  
Right Turn Percentage: 40%



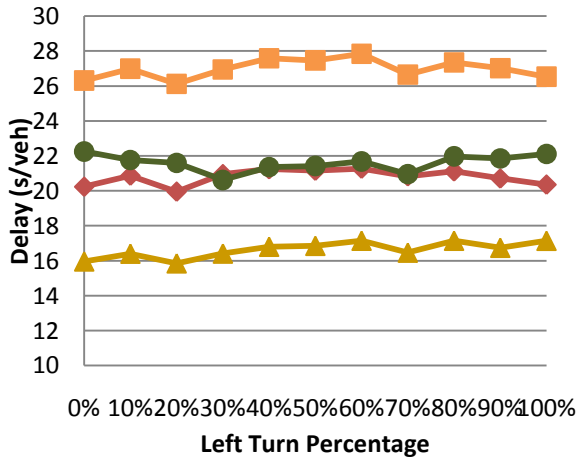
(b) Demand vs. Delay  
Left Turn Percentage: 80%  
Right Turn Percentage: 10%



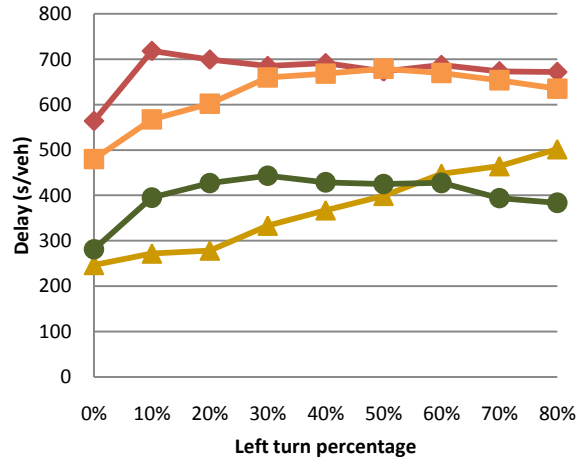
(c) Right Turn Percentage vs. Delay  
Demand: 500 veh/hr/approach  
Left Turn Percentage: 0%



(d) Right Turn Percentage vs. Delay  
Demand: 1600 veh/hr/approach  
Left Turn Percentage: 0%



(e) Left Turn Percentages vs. Delay  
Demand: 200 veh/hr/approach  
Right Turn Percentage: 0%



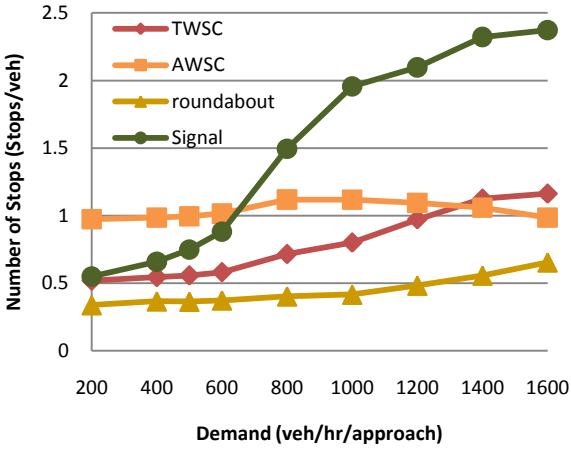
(f) Left Turn Percentages vs. Delay  
Demand: 1400 veh/hr/approach  
Right Turn Percentage: 20%

Figure 3-7: Delay graphs; Demands are in veh/hr/approach, Delay is in s/veh

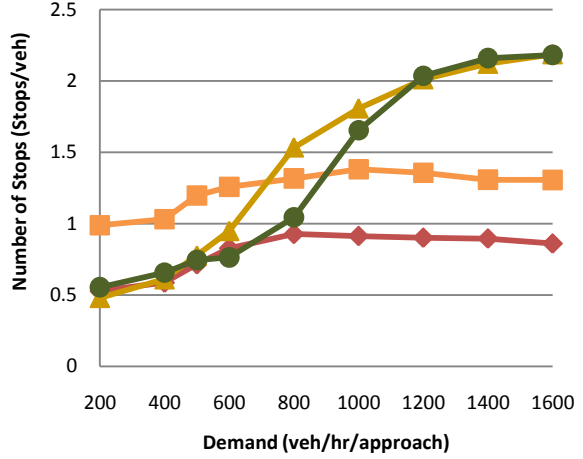


Left	right	200	400	500	600	800	1000	1200	1400	1600	
0%	0%	3	3	3	1	1	1	1	1	1	
	10%	3	3	3	1	1	1	1	1	1	
	20%	3	3	3	3	1	1	1	1	1	
	30%	3	3	3	3	1	1	1	1	1	
	40%	3	3	3	3	3	1	1	1	1	4 Signal
	50%	3	3	3	3	3	1	1	1	1	3 RA
	60%	3	3	3	3	3	3	1	1	1	2 AWSC
	70%	3	3	3	3	3	3	3	1	1	1 TWSC
	80%	3	3	3	3	3	3	3	3	1	
	90%	3	3	3	3	3	3	3	3	1	
	100%	3	3	3	3	3	3	3	3	1	
10%	0%	3	3	3	1	1	1	1	1	1	
	10%	3	3	3	1	1	1	1	1	1	
	20%	3	3	3	3	1	1	1	2	1	
	30%	3	3	3	3	1	1	1	1	1	
	40%	3	3	3	3	3	1	1	2	1	
	50%	3	3	3	3	3	3	1	2	1	
	60%	3	3	3	3	3	3	3	2	1	
	70%	3	3	3	3	3	3	3	3	1	
	80%	3	3	3	3	3	3	3	3	1	
	90%	3	3	3	3	3	3	3	3	1	
20%	0%	3	3	3	1	1	1	1	1	1	
	10%	3	3	3	1	1	1	1	1	1	
	20%	3	3	3	3	1	1	1	1	1	
	30%	3	3	3	3	1	1	1	1	1	
	40%	3	3	3	3	3	1	1	1	1	
	50%	3	3	3	3	3	1	1	2	1	
	60%	3	3	3	3	3	3	3	2	1	
	70%	3	3	3	3	3	3	3	2	1	
	80%	3	3	3	3	3	3	3	3	1	
30%	0%	3	3	3	1	1	1	1	1	1	
	10%	3	3	3	1	1	1	1	1	1	
	20%	3	3	3	3	1	1	1	1	1	
	30%	3	3	3	3	1	1	1	1	1	
	40%	3	3	3	3	1	1	1	1	1	
	50%	3	3	3	3	3	1	1	1	1	
	60%	3	3	3	3	3	1	1	2	1	
	70%	3	3	3	3	3	3	2	2	1	
40%	0%	3	3	3	1	1	1	1	1	1	
	10%	3	3	3	1	1	1	1	1	1	
	20%	3	3	3	3	1	1	1	1	1	
	30%	3	3	3	3	1	1	1	1	1	
	40%	3	3	3	3	1	1	1	1	1	
	50%	3	3	3	3	3	1	1	2	1	
	60%	3	3	3	3	3	1	2	2	1	
50%	0%	3	3	3	1	1	1	1	1	1	
	10%	3	3	4	1	1	1	1	1	1	
	20%	3	3	3	1	1	1	1	1	1	
	30%	3	3	3	1	1	1	1	1	1	
	40%	3	3	3	3	1	1	1	1	1	
	50%	3	3	3	3	1	1	1	2	1	
60%	0%	3	4	4	4	1	1	1	1	1	
	10%	3	3	4	1	1	1	1	1	1	
	20%	3	3	3	1	1	1	1	1	1	
	30%	3	3	3	1	1	1	1	1	1	
	40%	3	3	3	1	1	1	1	1	1	
70%	0%	3	4	4	4	1	1	1	1	1	
	10%	3	4	4	1	1	1	1	1	1	
	20%	3	1	4	1	1	1	1	1	1	
	30%	3	1	1	1	1	1	1	1	1	
80%	0%	1	4	4	4	1	1	1	1	1	
	10%	1	1	4	1	1	1	1	1	1	
	20%	1	1	4	1	1	1	1	1	1	
90%	0%	1	1	4	4	1	1	1	1	1	
	10%	1	1	1	1	1	1	1	1	1	
100%	0%	1	1	4	4	1	1	1	1	1	

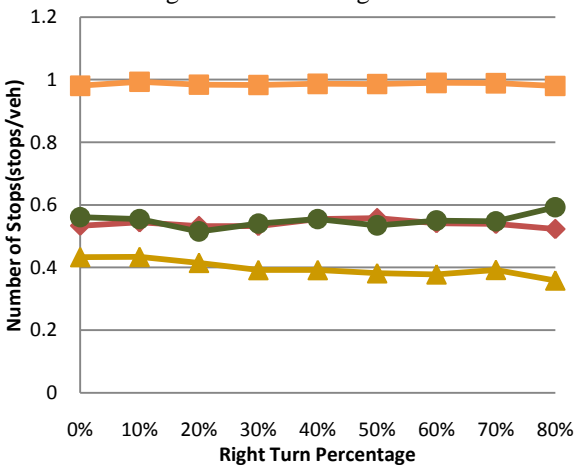
Figure 3-8: Control with Minimum Number of Vehicle Stops Grouped by Total Demand and Demand Configuration (Demands are in veh/hr/approach)



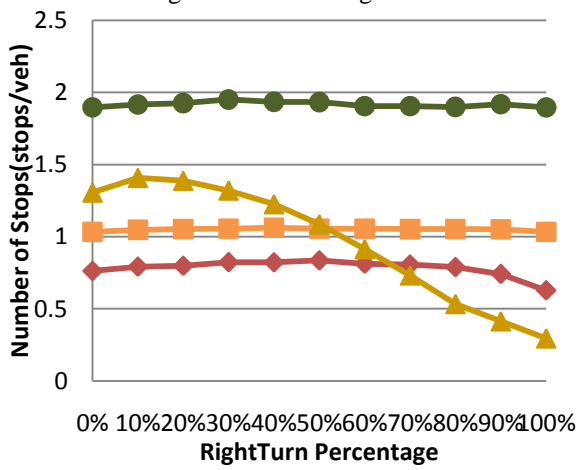
(a) Demand vs. Number of Stops  
Left Turn Percentage: 10%  
Right Turn Percentage: 90%



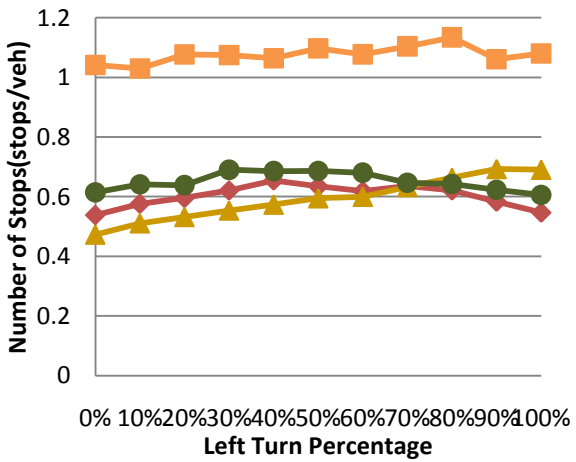
(b) Demand vs. Number of Stops  
Left Turn Percentage: 80%  
Right Turn Percentage: 10%



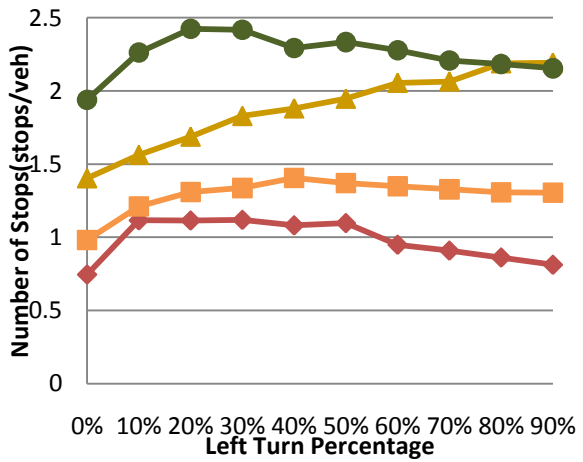
(c) Right Turn Percentage vs. Number of Stops  
Demand: 200 veh/hr/approach  
Left Turn Percentage: 20%



(d) Right Turn Percentage vs. Number of Stops  
Demand: 1400 veh/hr/approach  
Left Turn Percentage: 0%



(e) Left Turn Percentage vs. Number of Stops  
Demand: 400 veh/hr/approach  
Right Turn Percentage: 0%



(f) Left Turn Percentage vs. Number of Stops  
Demand: 1600 veh/hr/approach  
Right Turn Percentage: 10%

Figure 3-9: Variation in the Number of Vehicle Stops

# **Chapter 4: Are Roundabouts Environmentally Friendly? An Evaluation for Uniform Approach Demands**

Meredith Jackson and Hesham Rakha, submitted to the TRB for 2012

## **4.1 Abstract**

With the increased prevalence of roundabouts in the United States, there is a need to evaluate the environmental performance of roundabouts relative to other intersection control strategies. Studies have compared roundabouts with other intersection control strategies; however they are restricted to specific cases. These studies have suggested that when using environmental measures of effectiveness roundabouts can have few emissions and lower fuel consumption levels when compared to unsignalized intersections and can be better for lower demands than signalized intersections. However, some studies have not found this to be the case. In this study a generalized intersection with four single lane approaches with equal demand on all approaches was modeled to determine the control with the least fuel consumption, carbon dioxide, carbon monoxide, hydrocarbons and nitrogen oxide emissions. This study demonstrates that fuel consumption and CO<sub>2</sub> emissions depend upon turn demand and overall demand. Roundabouts can reduce fuel consumption and CO<sub>2</sub> emissions when left turn demands are lower than 30% of the overall demand or when left turn demand is less than 50% of the overall demand and right turn demand is greater than 10%. For most demands and turning ratios, roundabouts can also improve CO, HC and NO<sub>x</sub> emissions over traffic signal, two-way stop, and all-way stop control alternatives.

## **4.2 Introduction**

Today in the United States roundabouts are increasing in numbers. Many communities across the US have seen a new roundabout put in place by local municipalities to ease congestion or increase safety. [1] Roundabouts have been a part of the traffic system in other forms since 1905 when the first circular intersection was constructed in New York City. This intersection gave entering vehicles right of way which facilitated high speed entries but the intersection, referred to as a rotary had high congestion levels and high accident rates [1]. After the 1950's rotaries and other forms of circular intersections fell out of style mostly because of the negative experience with circular intersections of the time [1]. In 1966 the first modern roundabout was developed in the United Kingdom [1]. Rather than priority given to entering vehicles circulating vehicles were given right of way and entering and exiting traffic was channelized increasing safety and efficiency of circulating traffic.

A number of publications have dealt with how roundabouts compare to all-way stop control (AWSC), two-way stop control (TWSC), and signals using traditional measures of effectiveness such as delay, the number of stops and queue lengths. However, today's society wants more measures of effectiveness that deal with sustainability and the environment. Studies in this area have dealt with specific intersections and have found conflicting results when it comes to fuel consumption, carbon monoxide (CO) and dioxide (CO<sub>2</sub>), nitrogen oxides (NO<sub>x</sub>), and hydrocarbons (HC). This paper generalizes these studies for an intersection with single lane approaches with uniform demand across all approaches.

### 4.3 Literature Review

Many studies have discussed the comparison of the six basic MOEs. Some of these same studies briefly describe the change of a traditional intersection to a roundabout and how this change impacts fuel consumption and emissions. One author, Garder, suggests that due to the reductions in stops and delay there should be an overall reduction in emissions and fuel consumption as long as there is a similar amount of traffic from the minor street when the intersection is designed to have 18 mph (29 km/h). [15]

Unlike Garder, other study conclusions varied from small reductions and increases in fuel consumption and emissions when using a roundabout over other alternatives to large reductions in fuel consumption and emission levels. One such study proposes that when comparing to an unsignalized intersection, the roundabout increases emissions by 4% in nitrogen oxides (NO<sub>x</sub>) and 6% in carbon dioxide (CO<sub>2</sub>) and when compared to pre-timed signals, they decrease NO<sub>x</sub> by 21% and CO<sub>2</sub> by 29% [17]. This is also suggested by Høglund when his study explained that the uniform speeds around a roundabout would allow for a reduction in vehicle fuel consumption levels relative to a stop or signalized intersection where most cars must stop. [18].

Similar results were found when comparing specific roundabouts with AWSC. In two different studies, Mandavilli et al. compared six intersections before and after a roundabout was constructed. They demonstrated there was a reduction in all four MOEs when the peak hour volume was between 192 and 1220 veh/hr. In this case study hydrocarbons were reduced by 17 to 65% of emissions at the original intersection. Carbon dioxide was reduced by 21 to 42%, NO<sub>x</sub> was reduced by a 20 to 48%, and CO<sub>2</sub> was reduced by a 15 to 59%. [19 20]

Vlahos, Polus et al. found smaller improvements but just as impressive. Using SIDRA and demand data from a pair of roundabouts of 16350 AADT and 14000 AADT, They showed an overall reduction in delay, queues and all emissions for both a 1.00 Volume to Capacity ratio (V/C) and a 1.30 V/C they also showed with different turning movements and heavy vehicles the emissions also decreased. These researchers also go on to discuss the improvements of a roundabout over a signalized intersection. They give a table of cross over points where a roundabout is no longer better than a signal. [7]

However Ahn et al. showed that for a specific O-D demand the roundabout would actually cause more emissions than that of a stop or a signal controlled intersection. This O-D has a very low side street demand as compared to the main street. [8] This is contrary to most design guides which suggest that a more even demand for all approaches would allow for better flow around the roundabout.

### 4.4 Methodology

Several examples of comparing roundabouts and other intersection control have been conducted, however none of these can be really generalized and many have conflicting views as to whether signals or roundabouts are “better” for different traffic demand configurations. To start that general comparison, a simplified intersection should be considered. The intersection considered has four approaches each with only a single lane and no pocket lanes. Uniform flows between 200 veh/hr/approach and 1600 veh/hr/approach in increments of 200 veh/hr/approach and turn percentages between 0% and 100% in increments of 10% were used for the demands at the intersections. Four controls were considered: TWSC, AWSC, roundabout and traffic signal. The design speed for the intersection was 35 mph or 58 km/hr.

In earlier publications, SIDRA, VISSIM or NETSIM were used to conduct the analysis, however in this paper the INTEGRATION software is used because the software has been

validated against field data. An earlier study compared INTEGRATION and VISSIM results to field data for a basic intersection. The study concluded that the INTEGRATION output was more consistent with the queue length field observations at a TWSC intersection. The results demonstrated that VISSIM queue lengths were slightly longer than the field observations. The queue length is shown and INTEGRATION has a short queue length for the TWSC, while slightly longer queue length for signals and roundabouts. The results indicate that the optimal travel time is still similar even though there are slight differences between VISSIM and INTEGRATION [8].

The INTEGRATION simulation runs were made for the peak 15 minutes for each scenario considering a uniform demand. Approaches set to be 3 km in length to ensure that any queues would not spill back beyond the simulation geometric boundaries. A Saturation flow rate of 1800 veh/hr was assumed for all intersection alternatives. All simulations were executed to clear all vehicles that entered the network.

In a previous paper, traffic signal delay was experimented and resulted in a comparison of three timing schemes: one optimized by Webster Cobb method that resulted in a 180s four phase signal scheme in oversaturation, one with a 4 phase 50s cycle length and finally a scheme with 2 phases and 30s. Minimizing delay determined which phasing plan used because current standards have mostly attempted to minimize delay and not other measures of effectiveness. Therefore, in oversaturated conditions, the signal plans used in the comparison against roundabouts and stop control alternatives is the signal with two phase and 30s cycle length because it has the least delay of all plans.[10]

The design of the roundabout has to be considered. Based on the literature [1] the maximum suggested speed for circulating traffic in a single lane roundabout should be no more than 25 mph (40 km/hr). This design speed is used for the circulating traffic in the roundabout so that the speed variation between the free-flow speeds on the approach roads is minimal. This design speed corresponds to a roundabout with a 40 m diameter [1].

## 4.5 Modeling of Vehicle Emissions

The INTEGRATION model computes a number of measures of effectiveness (MOEs), including the average speed; vehicle delay; person delay; fuel consumed; vehicle emissions of carbon dioxide (CO<sub>2</sub>), carbon monoxide (CO), hydrocarbons (HC), oxides of nitrogen (NO<sub>x</sub>), and particulate matter (PM) in the case of diesel engines; and the vehicle crash risk and severity.

The computation of deci-second speeds permits the steady-state fuel consumption rate for each vehicle to be computed each second on the basis of its current instantaneous speed and acceleration level [16-19, 27, 36]. The VT-Micro model was developed as a statistical model from experimentation with numerous polynomial combinations of speed and acceleration levels to construct a dual-regime model as described in Equation (1), where  $L_{i,j}$  are model regression coefficients at speed exponent  $i$  and acceleration exponent  $j$ ,  $M_{i,j}$  are model regression coefficients at speed exponent  $i$  and acceleration exponent  $j$ ,  $v$  is the instantaneous vehicle speed in kilometers per hour (km/h), and  $a$  is the instantaneous vehicle acceleration (km/h/s). These fuel consumption and emission models were developed using data that were collected on a chassis dynamometer at the Oak Ridge National Labs (ORNL), data gathered by the Environmental Protection Agency (EPA), and data gathered using an on-board emission measurement device (OBD). These data included fuel consumption and emission rate measurements (CO, HC, and NO<sub>x</sub>) as a function of the vehicle's instantaneous speed and acceleration levels. The VT-Micro fuel consumption and emission rates were found to be highly

accurate compared to the ORNL data, with coefficients of determination ranging from 0.92 to 0.99. A more detailed description of the model derivation is provided in the literature [37].

$$F(t) = \begin{cases} \exp\left(\sum_{i=1}^3 \hat{a}_{i,j} \sum_{j=1}^3 L_{i,j} v^i a^j\right) & \text{for } a \geq 0 \\ \exp\left(\sum_{i=1}^3 \hat{a}_{i,j} \sum_{j=1}^3 M_{i,j} v^i a^j\right) & \text{for } a < 0 \end{cases} \quad (1)$$

From a general point of view, the use of instantaneous speed and acceleration data for the estimation of energy and emission impacts of traffic improvement projects provide a major advantage over state-of-practice methods that estimate vehicle fuel consumption and emissions based exclusively on the average speed and number of vehicle-miles traveled by vehicles on a given transportation link.

## 4.6 Results

Four isolated intersection control strategies were evaluated in this study. These included traffic signal control, AWSC, TWSC, and roundabouts. The optimum traffic signal timings were computed using the Highway Capacity Manual (HCM) procedures. Five environmental measures of effectiveness were analyzed including: fuel consumption, CO<sub>2</sub>, CO, NO<sub>x</sub> and HC emissions. The intersection control strategy that produced the minimum fuel consumption or emissions was identified and summarized in tables. All tables follow the same coloring scheme shown in Figure 4-1, Figure 4-3, Figure 4-5, Figure 4-7 and Figure 4-9, namely: traffic signal, roundabout, AWSC, and TWSC alternatives are represented by green, yellow, orange and red, respectively. The rows are grouped by left turn percentages and columns by demand level in veh/hr/approach. General trends in emissions and fuel consumption are exemplified in Figure 4-2, Figure 4-4, Figure 4-6, Figure 4-8 and Figure 4-10. In these graphs, traffic signal, roundabout, AWSC and TWSC alternatives are represented by red, orange, gold and green lines, respectively.

### 4.6.1 Fuel Consumption

Figure 4-1 illustrates that the minimum fuel consumption can be achieved by a roundabout when left turn demand is below 40% of total demand and between 40% and 60% left turning vehicles and right turn demand is greater than 10% of total demand. Above 60% left turning demand or between 40% and 60% left turning traffic with less than 20% right turning demand, the traffic signal or AWSC alternatives minimize fuel consumption. Signals are optimal when the intersections are at or above capacity (greater than 400 veh/hr/approach) and TWSC when intersections are below capacity or less than 400 veh/hr/approach.

Fuel consumption generally increases with increasing demand. As seen in Figure 4-2. When right turn percentage is moderate and left turn percentages are low: in this case, 10% of vehicles turn left and 20% of vehicles turn right, fuel consumption for the roundabout is 0.5090 l/veh when demand is 200 veh/hr/approach and increase to 0.6646 l/veh when demand is 1600 veh/hr/approach. This is the smallest increase and optimal intersection alternative. The AWSC, TWSC and traffic signal alternatives increase fuel consumption from 0.5253 l/veh, 0.5110 l/veh, and 0.5123 l/veh, respectively at 200 veh/hr/approach to 0.8153 l/veh, 0.8794 l/veh, and 0.7302 l/veh, respectively at 1600 veh/hr/approach. For higher left turn percentages, in this case, 80% of traffic turns left and 10% of demand turns right, the roundabouts, signals and TWSC alternatives are comparable when demands are below 500 veh/hr/approach. At 200 veh/hr/approach fuel consumption are 0.5128 l/veh, 0.5123 l/veh, 0.5104 l/veh, and 0.5253 l/veh

for the roundabout, signal, TWSC and AWSC alternatives, respectively and increase to 0.8100 l/veh, 0.7020 l/veh, 0.8422 l/veh and 0.8500 l/veh, respectively at 1600 veh/hr/approach.

Unlike the demand trends, the right turn percentages have minimal effects on fuel consumption when the approach demand is low as in the case of 600 veh/hr/approach and no left turns. In this case, for the roundabout and traffic signal, fuel consumption decreases marginally from 0.5229 l/veh and 0.5261 l/veh when there is no right turning demand to 0.5123 l/veh and 0.5261 l/veh, respectively, when all traffic turns right. Fuel consumption at the TWSC generally decreases from 0.5343 l/veh when no vehicles turn right to 0.5208 l/veh when all vehicles turn. There is a maximum when 20% of traffic turns right. Fuel consumption at the AWSC generally decreases from 0.6016 l/veh when there is no right turning demand to 0.5354 l/veh when all demand turns right with the greatest decrease between 0% right turning traffic and 10% right turning traffic. When demand is higher, for example 1600 veh/hr/approach and no left turning traffic, the roundabout and TWSC both decrease fuel consumption from 0.7028 l/veh and 0.8112 l/veh when no vehicles turn right to 0.5549 l/veh and 0.6724 l/veh, respectively when all vehicles turn right. On the other hand, at the AWSC and traffic signal alternatives, right turns have no discernable effect on fuel consumption levels and these alternatives produce an average fuel consumption of 0.7643 l/veh and 0.6672 l/veh, respectively.

Unlike the effects that right turning traffic has on fuel consumption left turning traffic tends to increase fuel consumption with increasing left turn percentages for low demand (400 veh/hr/approach and 20% of demand turning right). When there is no demand for left turns fuel consumption is 0.514 l/veh, 0.5199 l/veh, 0.5305 l/veh and 0.516 l/veh for the roundabout, traffic signal, AWSC and TWSC alternatives, respectively, and increases to 0.5213 l/veh, 0.5169 l/veh, 0.5312 l/veh and 0.5181 l/veh, respectively, when 80% of traffic turns left. For over-saturated conditions, for example 1000 veh/hr/approach and 20% of vehicles turning left, at the roundabout, fuel consumption increases from 0.5426 l/veh when no demand turns left to 0.6188 l/veh when 80% of vehicles turn left. At the TWSC and AWSC alternatives, fuel consumption increases from 0.6345 l/veh and 0.6104 l/veh, respectively, when no demand turns left to 0.6841 l/veh and 0.6753 l/veh, respectively, when 60% of vehicles turn left and then decreases to 0.6842 l/veh and 0.6547 l/veh, respectively, when 80% of vehicles are turning left. The traffic signal has a similar effect: increasing fuel consumption from 0.5518 l/veh when no vehicles turn left to 0.5953 l/veh when 30% of vehicles turn left and decreasing to 0.5852 l/veh when 80% of vehicles turn left..

#### **4.6.2 CO<sub>2</sub> Emissions**

Given that CO<sub>2</sub> is highly correlated with fuel consumption, similar trends are observed. Specifically, CO<sub>2</sub> emissions, as seen in Figure 4-3, is minimized when a roundabout control is applied when left turn demand is below 40% of the total demand and between 40% and 60% left turning vehicles and right turn demand is greater than 10% of total demand. Above 60% left turning demand or between 40% and 60% left turning traffic with less than 20% right turning demand, the traffic signal or AWSC alternatives minimize fuel consumption. Signals are optimal when the intersections are at or above capacity (greater than 400 veh/hr/approach) and TWSC when intersections are below capacity or less than 400 veh/hr/approach. Finally minimum CO<sub>2</sub> emissions are achieved when right turning traffic is lower than 20% with the AWSC when the demand is less than 500 veh/hr/approach and the signal when the intersection is above 1000 veh/hr/approach.

As was the case with fuel consumption, CO<sub>2</sub> emissions generally increase with increasing demand, as illustrated in Figure 4-4. As was the case with fuel consumption, right turn percentages have minimal effects on CO<sub>2</sub> emissions. Unlike the effects that right turning traffic has on fuel consumption and CO<sub>2</sub> emissions left turning traffic tends to increase CO<sub>2</sub> emissions with increasing left turn percentages.

### 4.6.3 CO Emissions

In Figure 4-5, the intersections with the minimum CO emissions are represented. With CO emissions like CO<sub>2</sub> emissions or fuel consumption, roundabouts have the fewest CO emissions over most demand levels and turn percentages. However the traffic signal have the least CO for some demand levels greater than 600 veh/hr/approach left turn demands greater than 90% or demand greater than 600 veh/hr/approach, no left turn demand and 30% to 100% of demand turning right.

Figure 4-6 demonstrates some of the trends in CO emissions. When left and right turn percentage is low for example 20% of traffic turns left and 20% of traffic turns right, all alternatives' CO emissions decrease then increase with roundabouts producing the lowest emissions over all demands. Carbon Monoxide emissions for the AWSC, roundabout and traffic signal alternatives decrease from 12.491 g/veh, 8.023 g/veh and 9.774 g/veh, respectively at 200 veh/hr/approach to 10.826 g/veh, 7.705 g/veh and 9.286 g/veh, respectively at 600 veh/hr/approach and increase to 11.527 g/veh, 8.3444 g/veh and 9.715 g/veh, respectively at 1600 veh/hr/approach.

When demand is low as in the example of 500 veh/hr/approach and no left turning traffic, CO emissions for the TWSC and traffic signal alternatives increase from 9.351 g/veh and 8.255 g/veh, respectively when there is no right turning traffic to 9.614 g/veh and 9.077 g/veh, respectively when 50% of traffic turns right and decreases to 9.403 g/veh and 8.255 g/veh, respectively when all of traffic turns right. For the AWSC, CO emissions decrease from 12.524 g/veh when no vehicles turn right to 11.132 g/veh when 50% of traffic turns and increases to 11.765 g/veh when all traffic turns right. These curves are similar to when there is more traffic like when demand is 1600 veh/hr/approach and there is no left turning traffic however the high demand creates greater increases and decreases. The TWSC and traffic signal alternatives increase CO emissions from 10.044 g/veh, and 8.256 g/veh, respectively, when there are no right turns to 11.584 g/veh and 9.161 g/veh, respectively, at 50% right turning to 10.076 g/veh and 8.256 g/veh, respectively when all vehicles turn left. For the AWSC, CO emissions decrease from 11.576 g/veh when no vehicles turn right to 10.886 g/veh when 50% of traffic turns and increases to 11.599 g/veh when all traffic turns right. Carbon Monoxide emissions for the roundabout decrease from 8.341 g/veh at no right turns to 7.128 g/veh when all traffic turns right.

Like the effects of right turn demand on CO emissions, when demand is low, in this case 400 veh/hr/approach and CO emissions for the AWSC decrease from 12.365 g/veh when no vehicles turn left to 11.641 g/veh when 60% of vehicles turn left and then increases to 12.422 when all vehicles turn. For the TWSC, CO emissions have an average of 9.354 g/veh and for the roundabout, CO emissions marginally increase from 7.859 g/veh when no vehicles turn left to 8.1128 g/veh when all vehicles turn left. For the traffic signal, CO emissions are 8.760 g/veh when no vehicles turn and 8.745 g/veh when all vehicles turn and there is a maximum of 9.606 g/veh when 50% of demand is left turning. When demand is greater at 1600 with 10% of traffic turns right, Both the AWSC and roundabout alternatives have increases in CO emissions from 11.479 g/veh and 8.237 g/veh, respectively, when no vehicles turn left to 12.251 g/veh and



8.9787 g/veh, respectively, when 90% of vehicles turn left. For both the TWSC and traffic signal alternatives, CO emissions increase from 10.825 g/veh and 8.549 g/veh, respectively, when no traffic turns left to 11.223 g/veh and 9.887 g/veh, respectively, when 40% of traffic turn left and decreases to 9.771 g/veh and 8.592 g/veh, respectively, when 90% of traffic turn left.

#### 4.6.4 NO<sub>x</sub> Emissions

Unlike CO emissions where roundabouts and signals have the least emissions, signals, roundabouts, and TWSC have the minimum NO<sub>x</sub> emissions. As seen in Figure 4-7, for demands lower than 500 veh/hr/approach and right turn percentages lower than 10% or left turn percentages greater than 40% a TWSC can have the least NO<sub>x</sub> emissions. For demands higher than 600 veh/hr/approach and right turn percentages lower than 10% with no left turning traffic or left turn percentages greater than 60%, a signal can have the least NO<sub>x</sub> emissions. Otherwise Roundabouts are optimal.

The trends for NO<sub>x</sub> emissions are exemplified in Figure 4-8. Like CO emissions, with increasing demand all four alternatives generally have decreases in NO<sub>x</sub> emissions before increasing them. In the case of 40% right turns and 20% left turns, for the roundabout, NO<sub>x</sub> marginally decreases from 0.7030 g/veh at 200 veh/hr/approach to 0.6984 g/veh at 600 veh/hr/approach and marginally increases to 0.7816 g/veh at 1600 veh/hr/approach. The TWSC has a marginal decrease of NO<sub>x</sub> emissions from 0.7011 g/veh at 200 veh/hr/approach to 0.6950 g/veh at 500 veh/hr/approach and an increase to 0.8840 g/veh at 1600 veh/hr/approach with the greatest increase between 800 veh/hr/approach and 1200 veh/hr/approach. For the traffic signal, NO<sub>x</sub> emissions has an average of 0.7132 g/veh from 200 veh/hr/approach to 800 veh/hr/approach and then an increase from 0.7218 g/veh at 800 veh/hr/approach to 0.8064 g/veh at 1600 veh/hr/approach. For the AWSC, NO<sub>x</sub> emissions decrease from 0.7679 g/veh at 200 veh/hr/approach to 0.7510 g/veh at 600 veh/hr/approach and increase to 0.9177 g/veh at 1600 veh/hr/approach. For a higher left turn percentage of 70% and 30% right turns, there is a similar pattern. The TWSC has a decrease in NO<sub>x</sub> emissions from 0.7058 g/veh at 200 veh/hr/approach to 0.6991 g/veh at 400 veh/hr/approach and increase to 0.8919 g/veh at 1600 veh/hr/approach. NO<sub>x</sub> emissions for the traffic signal decrease from 0.7049 g/veh at 200 veh/hr/approach to 0.6979 g/veh at 600 veh/hr/approach and increase to 0.7991 g/veh at 1600 veh/hr/approach. For the roundabout, NO<sub>x</sub> emissions decrease from 0.7088 g/veh at 200 veh/hr/approach to 0.7069 g/veh at 400 veh/hr/approach and increase to 0.8118 g/veh at 1600 veh/hr/approach. The AWSC has a decrease from 0.7700 g/veh at 200 veh/hr/approach to 0.7503 g/veh at 500 veh/hr/approach and an increase to 0.8873 g/veh at 1600 veh/hr/approach.

Right turn percentage versus NO<sub>x</sub> emissions in low demands (i.e. 500 veh/hr/approach and no left turning vehicles) demonstrate similar patterns to previous MOEs. Roundabouts decrease emissions from 0.6983 g/veh when no traffic turns right to 0.6674 g/veh when all traffic turns right. For the traffic signal NO<sub>x</sub> increases from 0.6834 g/veh when no traffic turns right to 0.6980 g/veh when 50% of traffic turns right and decreases to 0.6834 g/veh when all of traffic turns. TWSC has no marginal increase or decrease and has average NO<sub>x</sub> emissions of 0.6866 g/veh. For the AWSC NO<sub>x</sub> emissions decrease from 0.7858 g/veh when there is no right turning demand to 0.7477 g/veh when half the traffic turns right and marginally increases to 0.7532 g/veh when all traffic turns right. When Demand is higher, for example 1600 vehicles with no left turning traffic, roundabouts decreases from 0.7821 g/veh when there is no demand for right turns to 0.5528 g/veh when all traffic turns right. The traffic signal has an increase from 0.7108 g/veh when no traffic turns right to 0.7372 g/veh when half of traffic turns right and decreases to

0.7108 g/veh when all traffic is turning right. For the TWSC, NO<sub>x</sub> emissions increase from 0.8092 g/veh when no traffic turns right to 0.8216 g/veh when 20% of traffic turns right and decreases to 0.7042 g/veh when all of traffic turns right. The AWSC has an increase from 0.8429 g/veh at 0% right turners to 0.8462 g/veh when 10% turn right and decreases to 0.8325 when 50% turn right and finally increases to 0.8429 g/veh when all traffic turns.

Unlike increasing right turn demand, at a demand of 400 veh/hr/approach and no right turning vehicles, increasing left turn demand increases NO<sub>x</sub> emissions for the roundabout from 0.7004 g/veh when no traffic turns left to 0.7254 g/veh at 100% left turning vehicles. The traffic signal and TWSC have increases in NO<sub>x</sub> emissions from 0.6903 g/veh and 0.6917 g/veh, respectively when there is no left turn demand to 0.7099 g/veh and 0.7026 g/veh, respectively, when 40% of vehicles turn left and decreases to 0.6902 g/veh and 0.6920 g/veh, respectively, when all demand turns left. AWSC has no discernable trends and has average NO<sub>x</sub> emissions of 0.7622 g/veh. With more demand in this case 1600 veh/hr/approach and 10% of demand turns right, the roundabout increases emissions from 0.7257 g/veh when there is no left turning demand to 0.8802 g/veh at 90% left turning demand. For both the traffic signal and TWSC alternatives NO<sub>x</sub> emissions first increase from 0.7223 g/veh and 0.8192 g/veh, respectively when no demand turns left to 0.8184 g/veh and 0.9215 g/veh, respectively, at 30% left turning demand and decrease to 0.7495 g/veh and 0.8350 g/veh, respectively when 90% of traffic turns left. There is a local maximum of 0.8184 g/veh and 0.91792 g/veh when 30% of vehicles turn. The AWSC increases NO<sub>x</sub> emissions from 0.8462 g/veh when no vehicles turn left to 0.9342 g/veh when 50% of vehicles turn left and decreases emissions to 0.9156 g/veh when 90% of traffic turns left.

#### **4.6.5 HC Emissions**

Like CO, Roundabouts generally optimize HC emissions. As seen in Figure 4-9, however traffic signals are optimal when 100% of demand is thru demand or demand is at least 800 veh/hr/approach and left turning percentage is greater than 70%, or less than 10% of demand is turning right and demand is greater than 1000 veh/hr/approach.

In Figure 4-10, trends for HC emissions are exemplified. Although there is a general increase in emissions with increasing demand, there is also a decrease in emissions with low demand for all alternative controls. For example, when there is no left turn demand and 80% of overall demand is turning right, the roundabout marginally decreases emissions from 0.4834 g/veh at 200 veh/hr/approach to 0.4766 g/veh at 500 veh/hr/approach and increases emission to 0.5365 g/veh at 1600 veh/hr/approach. Hydrocarbon emissions for the traffic signal decrease from 0.5451 g/veh when demand is 200 veh/hr/approach to 0.5088 g/veh when demand is 600 veh/hr/approach and increase to 0.6554 g/veh when demand is 1600 veh/hr/approach. For the TWSC, HC emissions marginally decrease from 0.5338 g/veh when demand is 200 to 0.5243 g/veh when demand is 500 veh/hr/approach and increase to 0.7313 g/veh when demand is 1600. The AWSC decreases emissions from 0.6324 g/veh at 200 veh/hr/approach to 0.5892 g/veh when demand is 600 veh/hr/approach and increases emissions to 0.8065 g/veh at 1600 veh/hr/approach. When left turn percentage is high (i.e. 70%) and right turn percentage is low (i.e. 10%), roundabouts marginally decrease emissions from 0.4928 g/veh at 200 veh/hr/approach to 0.4910 g/veh at 400 veh/hr/approach and increases emissions to 0.7663 g/veh when demand is 1600 veh/hr/approach. For both the traffic signal and TWSC alternatives, emissions decrease from 0.5405 g/veh and 0.5372 g/veh at 200 veh/hr/approach to 0.5264 g/veh and 0.5338 g/veh when demand is 400 and increases to 0.7091 g/veh and 0.8691 g/veh when

demand is 1600 veh/hr/approach. For the AWSC, emissions decrease from 0.6276 g/veh at 200 veh/hr/approach to 0.6017 g/veh when demand is 500 veh/hr/approach and increase to 0.9140 g/veh when demand is 1600 veh/hr/approach.

For low demand (i.e. 400 veh/hr/approach and no left turning demand), right turn percentage has marginal effects on HC emissions for the roundabout and TWSC alternative. Roundabouts decrease emissions from 0.4872 g/veh when there are no right turning traffic to 0.4770 g/veh when all traffic turns right and TWSC averages 0.5272 g/veh overall. For the traffic signal, HC emissions marginally increase from 0.5123 g/veh when there are no right turners to 0.5259 g/veh when half of the vehicles turn right and marginally decrease to 0.5123 g/veh when all demand right turns. The TWSC has the opposite trend. Hydrocarbon emissions for a TWSC decrease from 0.6222 g/veh when no traffic turns right to 0.5987 g/veh when 40% of traffic turns right and increase to 0.6144 g/veh when all of traffic turns. For higher demand, in this case 1400 veh/hr/approach and no left turning traffic, there is a greater decrease for the roundabout and TWSC from 0.6291 g/veh and 0.7645 g/veh when no demand turns right to 0.5062 g/veh and 0.6449 g/veh when all vehicles turn right. The traffic signal marginally increases HC emissions from 0.6018 g/veh when there are no right turners to 0.6270 g/veh when half of demand is turning right and marginally decreases to 0.6018 g/veh when all of the demand turns right. The AWSC marginally decreases HC emissions from 0.76487 g/veh when no demand is right turning to 0.7478 g/veh when 50% of demand turns right and marginally increases HC emissions to 0.7647 g/veh when all of traffic turns right.

Similar to right turn demand's effects on HC emissions, at low demand (i.e. 400 veh/hr/approach) with no right turning demand, the roundabout and TWSC alternatives have marginal effects on HC emissions. The roundabouts have a marginal increase in emissions from 0.4872 g/veh at no left turning demand to 0.4989 g/veh when all traffic turns left and TWSC has an average HC emission of 0.5330 g/veh for all turn percentages. For the traffic signal, HC emissions increase from 0.5123 g/veh when there are no right turners to 0.5379 g/veh when half of the vehicles turns right and decrease to 0.5117 g/veh when all demand is right turning. The TWSC has the opposite trend. Hydrocarbon emissions for a TWSC decrease with noise from 0.6222 g/veh when no traffic turns right to 0.6040 g/veh when 40% of traffic turns right and increase to 0.6254 g/veh when all of traffic turns. With higher demand, for example 800 veh/hr/approach and 20% right turning demand, the roundabout has a larger difference between no left turning traffic and all traffic turning left. The roundabout increases HC emissions from 0.4939 g/veh when no demand turns left to 0.5443 g/veh at 80% left turning demand. For the traffic signal HC emissions increase from 0.5144 g/veh when no traffic is turning left to 0.5629 g/veh at 30% left turners and decrease to 0.5144 g/veh when 80% of the demand is left turners. The TWSC and AWSC alternatives increase HC emissions from 0.5783 g/veh and 0.6139 g/veh when no demand is turning left to 0.6330 g/veh and 0.6491 g/veh when 60% of demand turns left and decreases to 0.6159 g/veh and 0.6395 g/veh at 80% left turning traffic.

## **4.7 Conclusions and Recommendations**

In this study five environmental MOEs were used: fuel consumption, CO<sub>2</sub>, CO, NO<sub>x</sub> and HC emissions. Both CO<sub>2</sub> emissions and fuel consumption had similar results. Below 500 veh/hr/approach, the TWSC minimized fuel consumption and CO<sub>2</sub> emissions when left turn demand was greater than 50% of the total demand. The traffic signal minimized fuel consumption and CO<sub>2</sub> emissions when the left turn demand was greater than 50% and total demand was greater than 500 veh/hr/approach, and when total demand was less than 500

veh/hr/approach and right turn demand was less than 10%, CO<sub>2</sub> emissions are also minimized by a traffic signal. Otherwise roundabouts minimized fuel consumption and CO<sub>2</sub> emission levels compared to other intersection control alternatives. The traffic signal minimized both CO and HC emissions when left turn demand was low or high and roundabouts minimized both emissions for all other demand configurations. Finally NO<sub>x</sub> was minimized by the traffic signal control for demands over 500 veh/hr/approach when the left turn demand was greater than 50% and by the TWSC under 500 veh/hr/approach when the left turn demand was greater than 50% or right turn demand was less than 20%. Otherwise a roundabout was optimal for NO<sub>x</sub> emissions.

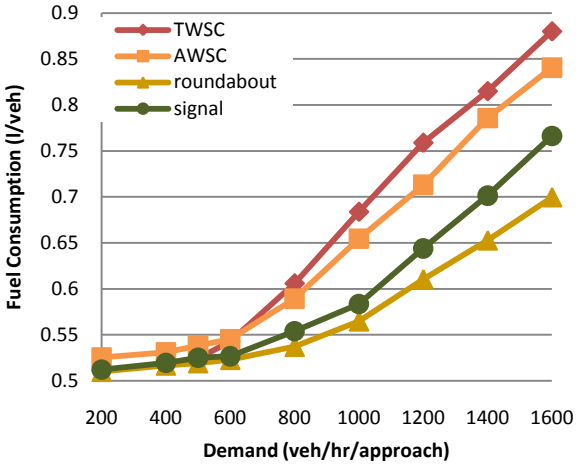
Roundabouts can minimize all five environmental impacts studied in certain situations. More research is needed to consider other factors. More demands with more independence between approaches is necessary to characterize the full impact of a roundabout especially when the demand is not equal on all approaches. This research dealt with a very specific geometry and speed limit. Consequently, the analysis should be extended to consider different geometries including more lanes and different sizes of roundabouts and speeds in order to generalize these findings to a larger domain of traffic conditions.

## 4.8 References

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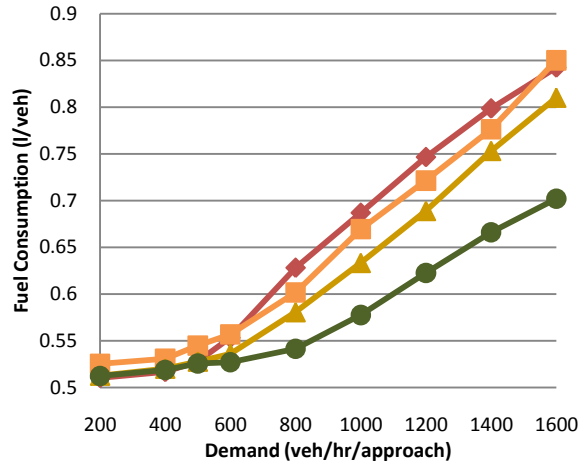
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Figure 4-1: Control with minimum Fuel Consumption grouped by Demand and Demand Configurations (Demand in veh/hr/approach)



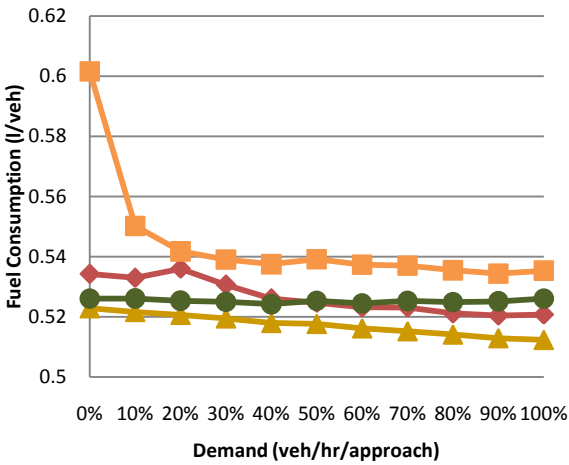
(a) Demand vs. Fuel

Left turn percentage: 20%  
Right Turn Percentage: 10%



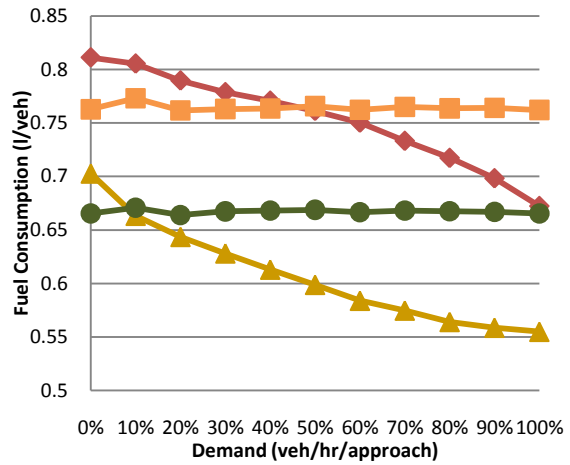
(b) Demand vs. Fuel

Left turn percentage: 80%  
Right Turn Percentage: 10%



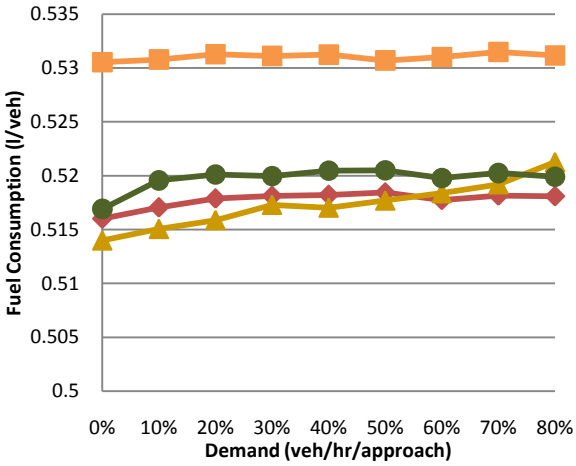
(c) Right Turn Percentage vs. Fuel

Demand: 600 veh/hr/approach  
Left Turn Percentage: 0%



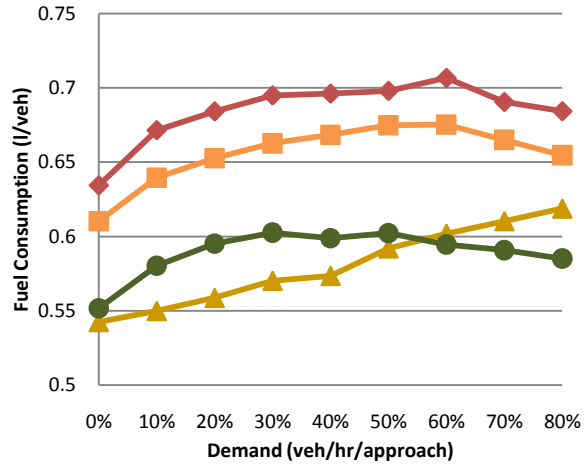
(d) Right Turn Percentage vs. Fuel

Demand: 1600 veh/hr/approach  
Left Turn Percentage: 0%



(e) Left Turn Percentage vs. Fuel

Demand: 400 veh/hr/approach  
Right Turn Percentage: 20%



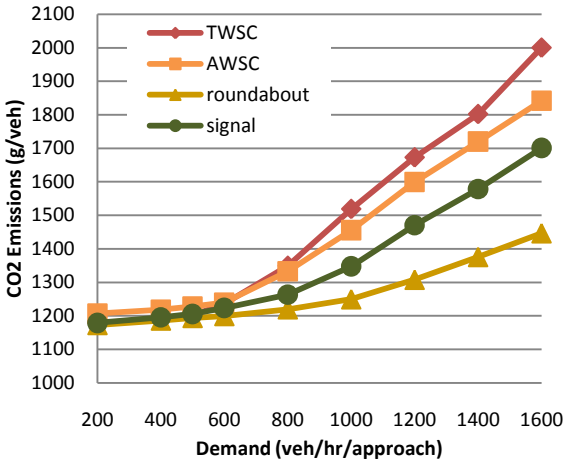
(f) Left Turn Percentage vs. Fuel

Demand: 1000 veh/hr/approach  
Right Turn Percentage: 20%

Figure 4-2: Variations in Fuel Consumption (l/veh)

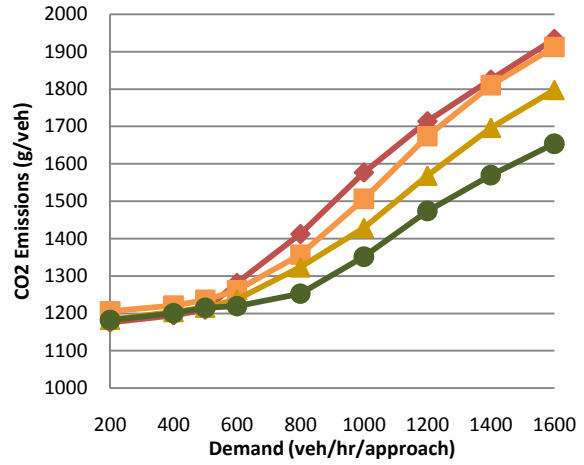
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	90%	3	3	3	3	3	3	3	3	3	
20%	0%	1	4	3	3	3	4	4	4	4	
	10%	3	1	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
	70%	3	3	3	3	3	3	3	3	3	
	80%	3	3	3	3	3	3	3	3	3	
30%	0%	4	4	4	4	4	4	4	4	4	
	10%	4	1	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
	70%	3	3	3	3	3	3	3	3	3	
	80%	3	3	3	3	3	3	3	3	3	
40%	0%	1	4	4	4	4	4	4	4	4	
	10%	4	1	4	3	3	4	3	3	4	
	20%	1	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
50%	0%	1	4	4	4	4	4	4	4	4	
	10%	4	1	4	4	4	4	4	4	4	
	20%	3	1	3	3	4	3	3	3	3	
	30%	1	3	3	3	3	3	3	3	3	
	40%	3	1	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
60%	0%	1	4	4	4	4	4	4	4	4	
	10%	1	1	4	4	4	4	4	4	4	
	20%	1	1	4	4	4	4	4	3	4	
	30%	1	1	3	3	4	3	3	3	3	
	40%	1	1	3	3	3	3	3	3	3	
70%	0%	1	4	4	4	4	4	4	4	4	
	10%	1	4	4	4	4	4	4	4	4	
	20%	1	1	4	4	4	4	4	4	4	
	30%	1	1	1	4	4	3	4	3	4	
80%	0%	1	4	4	4	4	4	4	4	4	
	10%	1	1	4	4	4	4	4	4	4	
	20%	1	1	1	4	4	4	4	4	4	
90%	0%	1	1	4	4	4	4	4	4	4	
	10%	1	1	1	4	4	4	4	4	4	
100%	0%	1	1	4	4	4	4	4	4	4	

Figure 4-3: Control with minimum CO<sub>2</sub> grouped by Grouped by Demand and Demand Configuration (Demands are in veh/hr/approach)



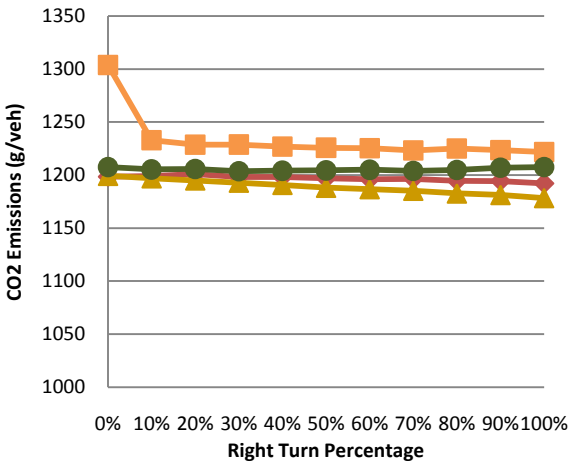
(a) Demand vs. CO<sub>2</sub>

Left turn percentage: 10%  
Right Turn Percentage: 40%



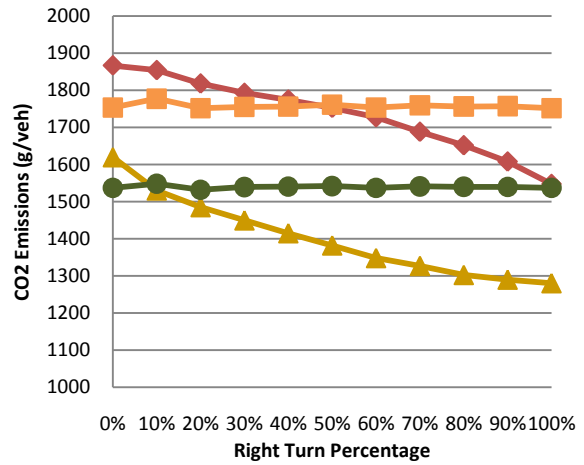
(b) Demand vs. CO<sub>2</sub>

Left turn percentage: 80%  
Right Turn Percentage: 20%



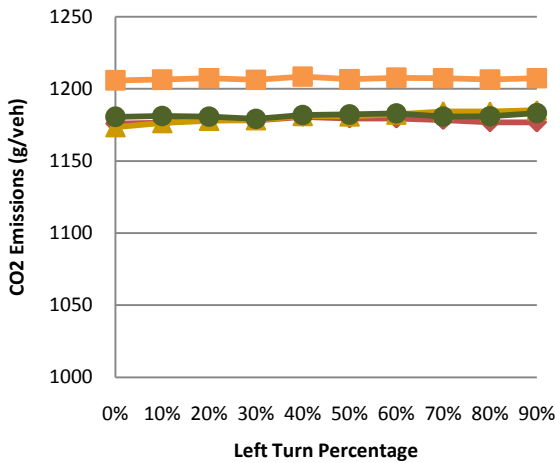
(c) Right Turn Percentage vs. CO<sub>2</sub>

Demand: 500 veh/hr/approach  
Left Turn Percentage: 0%



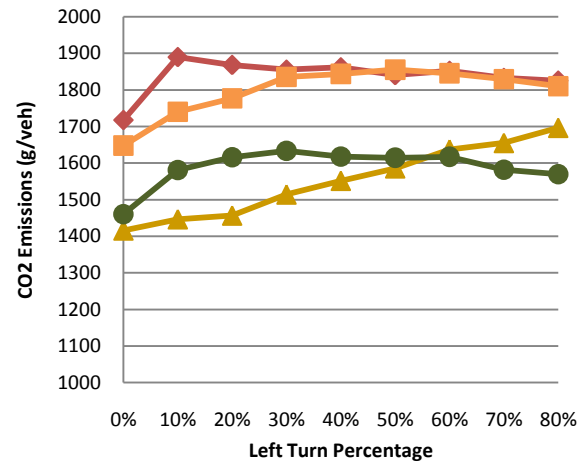
(d) Right Turn Percentage vs. CO<sub>2</sub>

Demand: 1600 veh/hr/approach  
Left Turn Percentage: 0%



(e) Left Turn Percentage vs. CO<sub>2</sub>

Demand: 200 veh/hr/approach  
Right Turn Percentage: 10%



(f) Left Turn Percentage vs. CO<sub>2</sub>

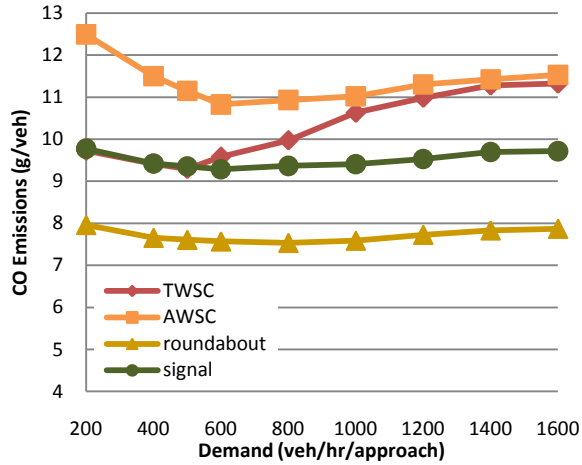
Demand: 1400 veh/hr/approach  
Right Turn Percentage: 20%

Figure 4-4: Variations in CO<sub>2</sub> Emissions (g/veh)



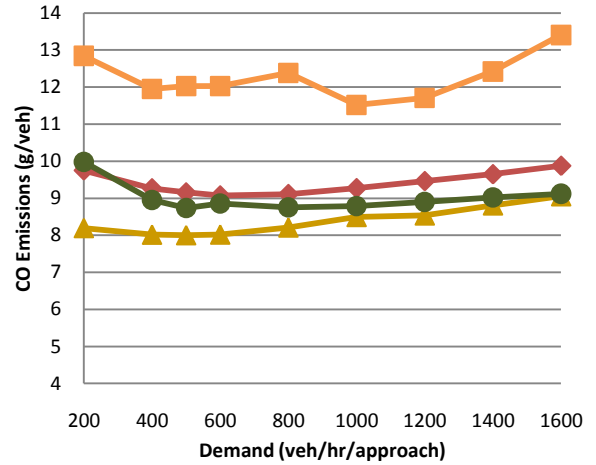
Left	right	200	400	500	600	800	1000	1200	1400	1600	
0%	0%	3	3	3	3	3	3	3	3	3	
	10%	3	3	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	4 Signal
	40%	3	3	3	3	3	3	3	3	3	3 RA
	50%	3	3	3	3	3	3	3	3	3	2 AWSC
	60%	3	3	3	3	3	3	3	3	3	1 TWSC
	70%	3	3	3	3	3	3	3	3	3	
	80%	3	3	3	3	3	3	4	3	3	
	90%	3	3	3	3	4	4	4	4	4	
	100%	3	3	3	4	4	4	4	4	4	
10%	0%	3	3	3	3	3	3	3	3	3	
	10%	3	3	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
	70%	3	3	3	3	3	3	3	3	3	
	80%	3	3	3	3	3	3	3	3	3	
	90%	3	3	3	3	3	3	3	3	3	
20%	0%	3	3	3	3	3	3	3	3	3	
	10%	3	3	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
	70%	3	3	3	3	3	3	3	3	3	
	80%	3	3	3	3	3	3	3	3	3	
30%	0%	3	3	3	3	3	3	3	3	3	
	10%	3	3	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
	70%	3	3	3	3	3	3	3	3	3	
40%	0%	3	3	3	3	3	3	3	3	3	
	10%	3	3	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
50%	0%	3	3	3	3	3	3	3	3	3	
	10%	3	3	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
60%	0%	3	3	3	3	3	3	3	3	3	
	10%	3	3	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
70%	0%	3	3	3	3	3	3	3	3	3	
	10%	3	3	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
80%	0%	3	3	3	3	3	3	3	3	3	
	10%	3	3	3	3	3	3	4	3	3	
	20%	3	3	3	3	3	4	3	4	4	
90%	0%	3	3	3	4	3	3	4	4	4	
	10%	3	3	4	4	4	4	4	4	4	
100%	0%	4	4	3	4	4	4	4	4	4	

Figure 4-5: Control with minimum CO grouped by Demand and Demand Configuration (Demands are in veh/hr/approach)



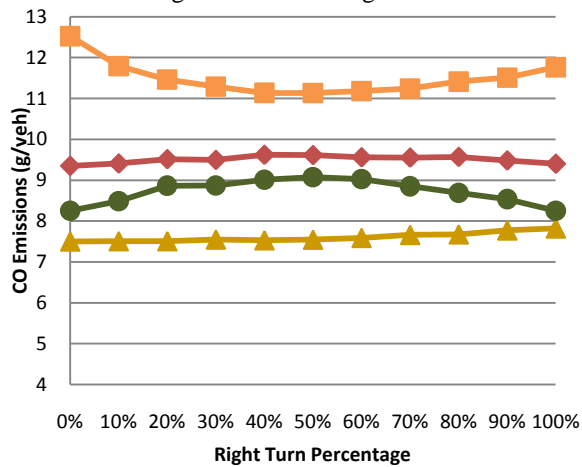
(a) Demand vs. CO

Left turn percentage: 20%  
Right Turn Percentage: 20%



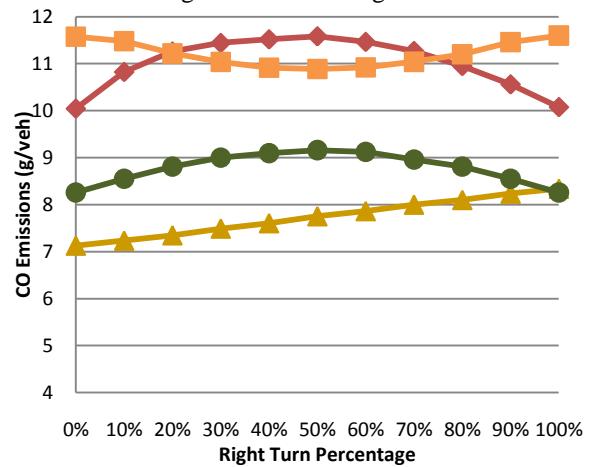
(b) Demand vs. CO

Left turn percentage: 80%  
Right Turn Percentage: 0%



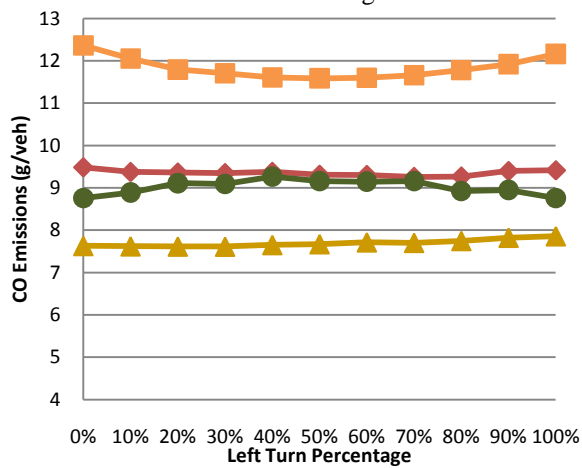
(c) Right Turn Percentage vs. CO

Demand: 500 veh/hr/approach  
Left Turn Percentage: 0%



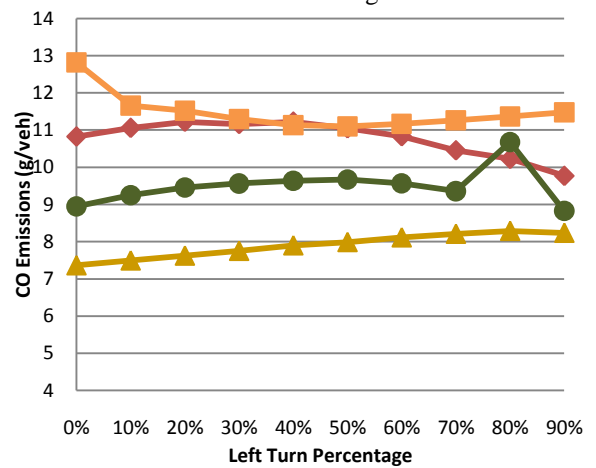
(d) Right Turn Percentage vs. CO

Demand: 1600 veh/hr/approach  
Left Turn Percentage: 0%



(e) Left Turn Percentage vs. CO

Demand: 400 veh/hr/approach  
Right Turn Percentage: 0%



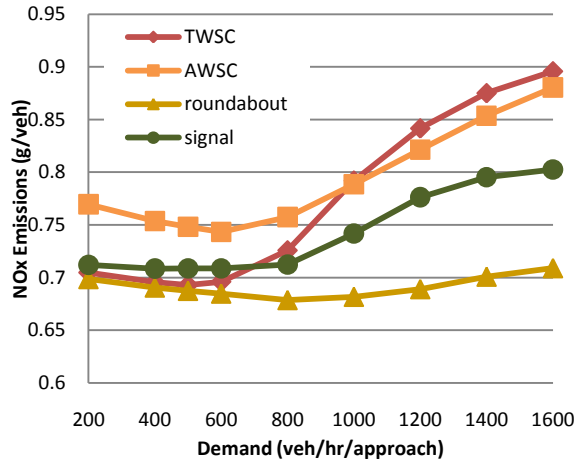
(f) Left Turn Percentage vs. CO

Demand: 1600 veh/hr/approach  
Right Turn Percentage: 10%

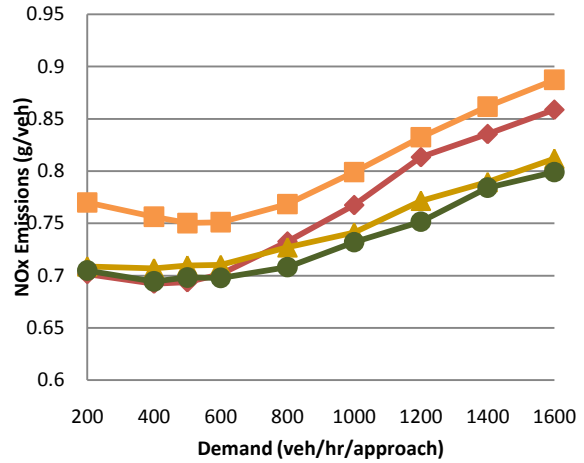
Figure 4-6: Variations in CO emissions (g/veh)

Left	right	200	400	500	600	800	1000	1200	1400	1600	
0%	0%	1	4	4	4	4	4	4	4	4	
	10%	1	1	4	4	4	4	4	4	4	
	20%	1	3	3	4	4	3	3	3	3	
	30%	3	3	3	3	4	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	4 Signal
	50%	3	3	3	3	3	3	3	3	3	3 RA
	60%	3	3	3	3	3	3	3	3	3	2 AWSC
	70%	3	3	3	3	3	3	3	3	3	1 TWSC
	80%	3	3	3	3	3	3	3	3	3	
	90%	3	3	3	3	3	3	3	3	3	
100%	3	3	3	3	4	3	3	3	3		
10%	0%	1	1	4	4	4	3	3	3	3	
	10%	1	1	1	4	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
	70%	3	3	3	3	3	3	3	3	3	
	80%	3	3	3	3	3	3	3	3	3	
	90%	3	3	3	3	3	3	3	3	3	
20%	0%	1	1	3	4	3	3	3	3	3	
	10%	1	1	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
	70%	3	3	3	3	3	3	3	3	3	
	80%	3	3	3	3	3	3	3	3	3	
30%	0%	1	1	1	4	3	3	3	3	3	
	10%	1	1	3	3	3	3	3	3	3	
	20%	3	1	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
	70%	3	3	3	3	3	3	3	3	3	
40%	0%	1	1	3	4	3	4	4	3	4	
	10%	1	1	3	3	3	3	3	3	3	
	20%	1	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	1	1	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
50%	0%	1	1	4	4	4	4	4	4	4	
	10%	1	1	1	4	4	4	4	4	4	
	20%	3	1	3	3	3	3	3	3	3	
	30%	1	1	3	3	3	3	3	3	3	
	40%	3	1	1	3	3	3	3	3	3	
	50%	3	1	1	3	3	3	3	3	3	
60%	0%	1	1	4	4	4	4	4	4	4	
	10%	1	1	1	4	4	4	4	4	4	
	20%	1	1	1	4	4	4	3	4	4	
	30%	1	1	1	4	4	3	3	3	3	
	40%	1	1	1	3	3	3	3	3	3	
70%	0%	1	1	4	4	4	4	4	4	4	
	10%	1	1	1	4	4	4	4	4	4	
	20%	1	1	1	4	4	4	4	4	4	
	30%	1	1	1	4	4	4	4	4	4	
80%	0%	1	1	4	4	4	4	4	4	4	
	10%	1	4	4	4	4	4	4	4	4	
	20%	1	4	4	4	4	4	4	4	4	
90%	0%	1	4	4	4	4	4	4	4	4	
	10%	1	4	4	4	4	4	4	4	4	
100%	0%	1	4	4	4	4	4	4	4	4	

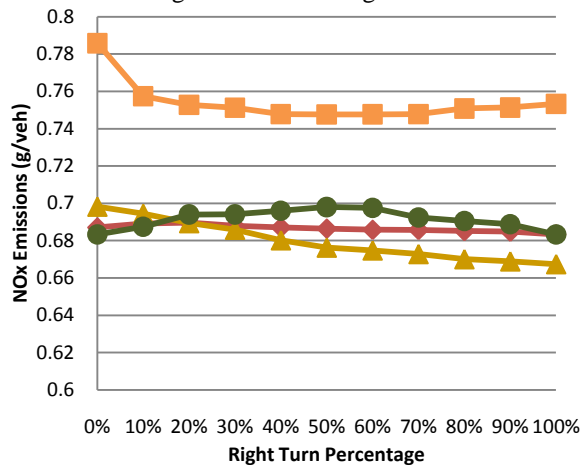
Figure 4-7: Control with minimum NO<sub>x</sub> grouped by Demand and Demand Configuration (Demands are in veh/hr/approach)



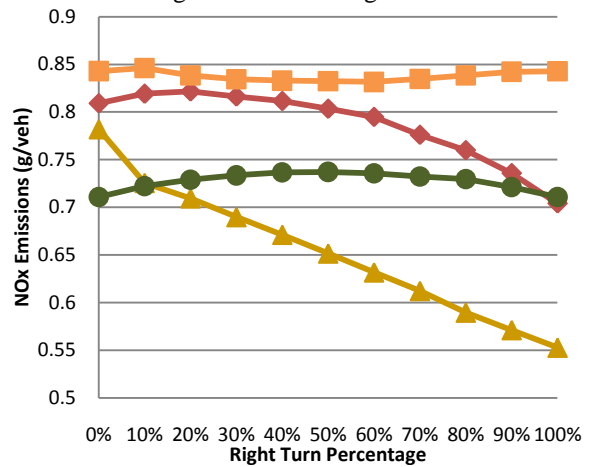
(a) Demand vs. NO<sub>x</sub>  
Left turn percentage: 20%  
Right Turn Percentage: 40%



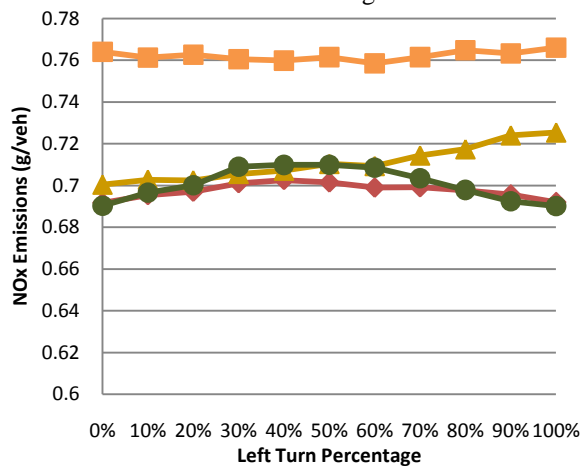
(b) Demand vs. NO<sub>x</sub>  
Left turn percentage: 70%  
Right Turn Percentage: 30%



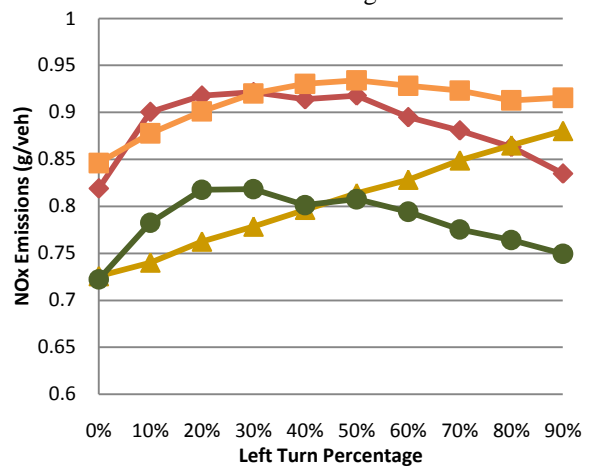
(c) Right Turn Percentage vs. NO<sub>x</sub>  
Demand: 500 veh/hr/approach  
Left Turn Percentage: 0%



(d) Right Turn Percentage vs. NO<sub>x</sub>  
Demand: 1600 veh/hr/approach  
Left Turn Percentage: 0%



(e) Left Turn Percentage vs. NO<sub>x</sub>  
Demand: 400 veh/hr/approach  
Right Turn Percentage: 0%

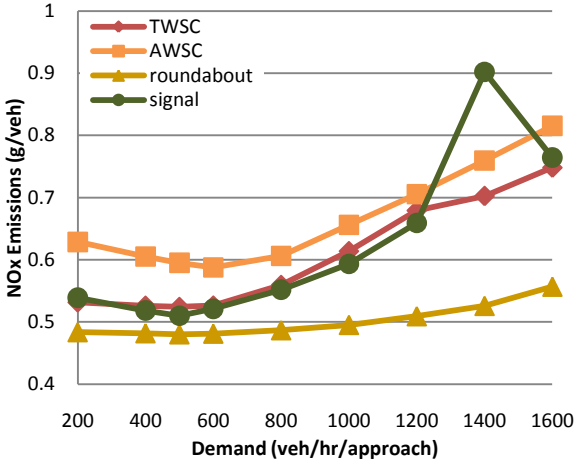


(f) Left Turn Percentage vs. NO<sub>x</sub>  
Demand: 1600 veh/hr/approach  
Right Turn Percentage: 10%

Figure 4-8: Variations in NO<sub>x</sub> Emissions (g/veh)

Left	right	200	400	500	600	800	1000	1200	1400	1600	
0%	0%	3	3	3	3	3	4	4	4	4	
	10%	3	3	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	4 Signal
	40%	3	3	3	3	3	3	3	3	3	3 RA
	50%	3	3	3	3	3	3	3	3	3	2 AWSC
	60%	3	3	3	3	3	3	3	3	3	1 TWSC
	70%	3	3	3	3	3	3	3	3	3	
	80%	3	3	3	3	3	3	3	3	3	
	90%	3	3	3	3	3	3	3	3	3	
	100%	3	3	3	3	3	3	3	3	3	
10%	0%	3	3	3	3	3	3	3	3	3	
	10%	3	3	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
	70%	3	3	3	3	3	3	3	3	3	
	80%	3	3	3	3	3	3	3	3	3	
	90%	3	3	3	3	3	3	3	3	3	
20%	0%	3	3	3	3	3	3	3	3	3	
	10%	3	3	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
	70%	3	3	3	3	3	3	3	3	3	
	80%	3	3	3	3	3	3	3	3	3	
30%	0%	3	3	3	3	3	3	3	3	3	
	10%	3	3	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
	70%	3	3	3	3	3	3	3	3	3	
40%	0%	3	3	3	3	3	3	3	3	3	
	10%	3	3	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
	60%	3	3	3	3	3	3	3	3	3	
50%	0%	3	3	3	3	3	3	4	3	3	
	10%	3	3	3	3	3	3	3	3	3	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
	50%	3	3	3	3	3	3	3	3	3	
60%	0%	3	3	3	3	3	4	4	4	4	
	10%	3	3	3	3	3	3	4	3	4	
	20%	3	3	3	3	3	3	3	3	3	
	30%	3	3	3	3	3	3	3	3	3	
	40%	3	3	3	3	3	3	3	3	3	
70%	0%	3	3	3	3	4	4	4	4	4	
	10%	3	3	3	3	3	4	4	4	4	
	20%	3	3	3	3	3	4	4	4	4	
	30%	3	3	3	3	3	3	3	3	3	
80%	0%	3	3	3	3	4	4	4	4	4	
	10%	3	3	3	3	4	4	4	4	4	
	20%	3	3	3	3	4	4	4	4	4	
90%	0%	3	3	3	4	4	4	4	4	4	
	10%	3	3	4	4	4	4	4	4	4	
100%	0%	4	4	3	4	4	4	4	4	4	

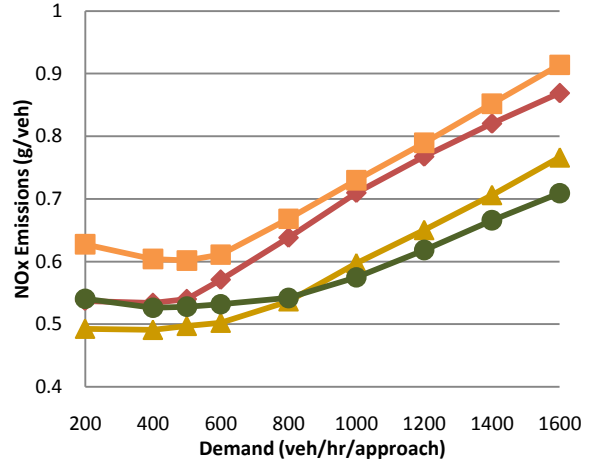
Figure 4-9: Control with minimum HC grouped by Total Demand and Demand Configuration (Demands are in veh/hr/approach)



(a) Demand vs. HC

Left turn percentage: 0%

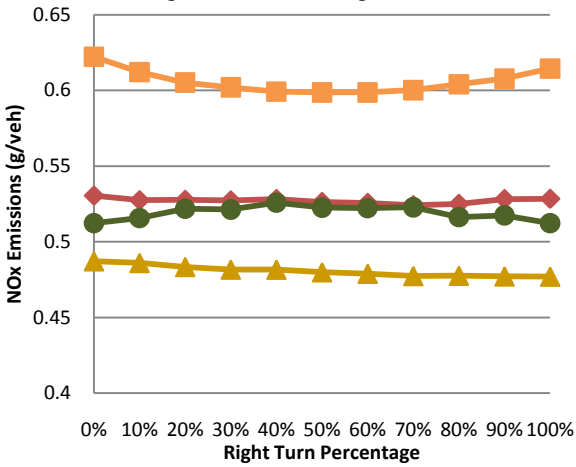
Right Turn Percentage: 80%



(b) Demand vs. HC

Left turn percentage: 70%

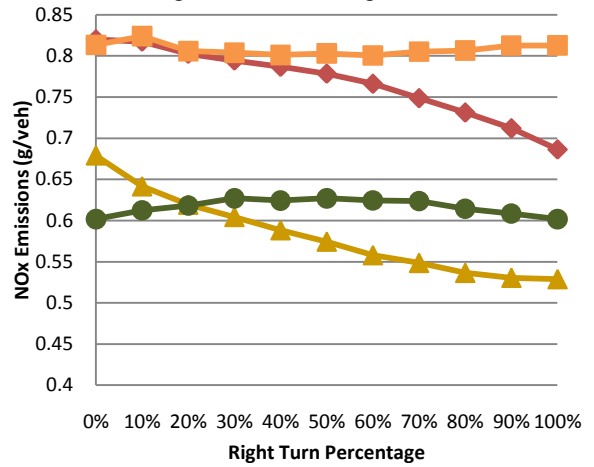
Right Turn Percentage: 10%



(c) Right Turn Percentage vs. HC

Demand: 400 veh/hr/approach

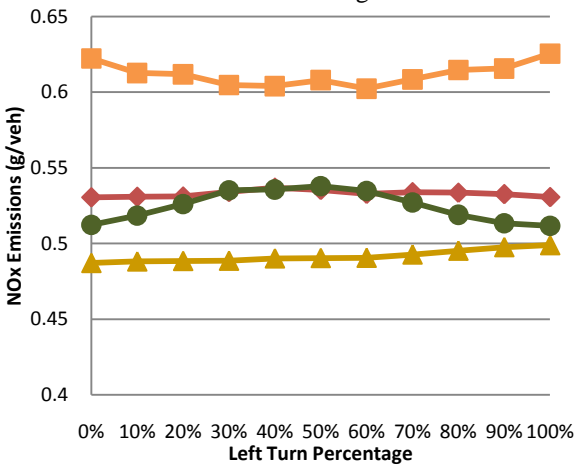
Left Turn Percentage: 0%



(d) Right Turn Percentage vs. HC

Demand: 1400 veh/hr/approach

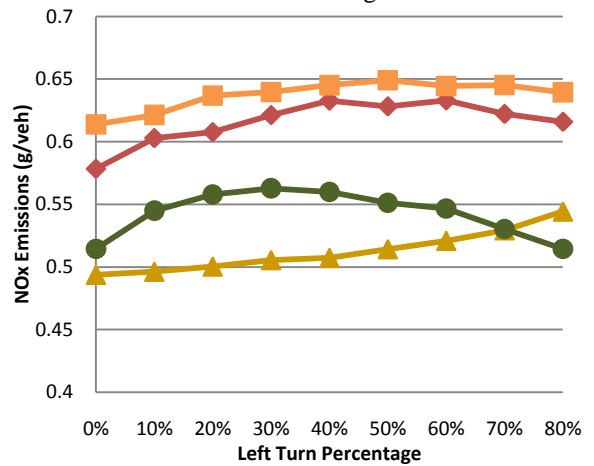
Left Turn Percentage: 0%



(e) Left Turn Percentage vs. HC

Demand: 400 veh/hr/approach

Right Turn Percentage: 0%



(f) Left Turn Percentage vs. HC

Demand: 800 veh/hr/approach

Right Turn Percentage: 20%

Figure 4-10: Variations in Hydrocarbon Emissions (g/veh)

## **Chapter 5: Conclusions and Recommendations**

### **5.1 Conclusions from Research**

In the literature there were many case studies in which specific conditions were compared for a roundabout, traffic signal and stop control. These studies were narrow in their focus of total approach demands and/or turn percentages. This research went beyond the narrow focus to get a wider comparison of average delay, number of stops, fuel consumption and carbon dioxide, carbon monoxide, nitrogen oxides and hydrocarbon at different alternatives with equal demand on all approaches.

In this work, two traditional MOEs were used to compare four alternative intersection control strategies (AWSC, TWSC, roundabouts, and traffic signals), namely: vehicle delay and stops. These comparisons were made considering a single lane approach with a 35 mph speed limit and equal demand on all approaches. Delay, the measure typically used in the literature, is minimized with the use of a roundabout intersection control for all demand levels below 500 veh/hr/approach. Above this point if the left turn percentage exceeds 70%, the traffic signal control becomes more efficient. The roundabout alternative also produces the fewest vehicle stops for low demand levels, low left turn demands, and high right turn demands; however, TWSC produces the least number of vehicle stops when the through demand and total demand is high. The results demonstrate that for low demand levels roundabouts should be a part of design alternatives considered for isolated intersection control. In all conditions, the intersection that can be considered optimal is very different depending upon the turn percentages for both left and right turners as well as the geometry and overall demand. Also, which measure of effectiveness to consider in the analysis may change the optimal intersection control for a given condition or set of conditions.

Five environmental MOEs were compared: fuel consumption and CO<sub>2</sub>, CO, NO<sub>x</sub> and HC emission. Carbon dioxide emissions and fuel consumption had similar results. Below 500 veh/hr/approach, the TWSC minimized fuel consumption and CO<sub>2</sub> emissions when left turn demand was greater than 50% of total demand. The traffic signal minimized fuel consumption and CO<sub>2</sub> emissions when left turn demand was greater than 50% and total demand was greater than 500 veh/hr/approach. When total demand is less than 500 veh/hr/approach and right turn demand is less than 10%, CO<sub>2</sub> emissions are also minimized by a traffic signal. Otherwise, roundabouts minimize fuel consumption and CO<sub>2</sub> emission as compared to the other alternatives. The traffic signal minimizes both CO emissions and HC emissions when left turn demand is low or high and roundabouts minimize both emissions the rest of the time. Finally, NO<sub>x</sub> is minimized by the traffic signal over 500 veh/hr/approach when left turn demand is greater than 50%, and by the TWSC under 500 veh/hr/approach when left turn demand is greater than 50% or right turn demand is less than 20%. Otherwise a roundabout is optimal for NO<sub>x</sub> emissions.

### **5.2 Further Research**

Roundabouts can optimize delay, number of stops and all 5 environmental impacts studied in certain situations. It will be important for planners, designers and others to consider roundabouts as an alternative. However, more research will be needed to see the bigger consequences of developing a roundabout in an area. For example, more data is needed reflecting different demands and more independence between approaches. This research also dealt with a very specific geometry and speed. Different geometry, including more lanes and different sizes of roundabouts and speeds would help to generalize these findings to a larger domain of traffic conditions. Finally, with all of this data and the greater generalization a model would provide designers a better estimate of which alternative is optimal for a specific site.