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Evaluation of Roundabouts versus Signalized and Unsignalized Intersections in Delaware

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Project Report

Prepared for

The Delaware Department of Transportation

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ABSTRACT AND EXECUTIVE SUMMARY

This study evaluates and compares a single-lane roundabout with an unsignalized (two-way stop controlled) intersection and a signalized (pre-timed) intersection and recommends conditions under which the construction of a roundabout may be more appropriate for an intersection. The measures of effectiveness used for the comparisons were effective intersection capacity, major and minor road entry lane capacity, major and minor road average delay, major and minor road queue length, and emission rates (CO, NO_X, HC, and CO_2). These measures are provided by the aaSIDRA software package which could then be used to establish "thresholds" in the major road one-way volume. These volumes indicate the threshold values for which the roundabout performs better than the unsignalized and signalized intersections.

Two single-lane roundabouts with medium to high traffic volumes were studied in order to obtain local driver gap acceptance characteristics for use in the SIDRA analyses. The roundabouts selected for study are located in Maryland, near its border with Delaware, and so the data collected is assumed to be applicable to Delaware drivers as well. Observations of these sites found critical gaps of 3.85 and 3.91 s and follow-up times of 1.9 and 2.1 s, both lower than those values recommended by the HCM. Analyses using the SIDRA software with a critical gap of 3.9 s and a follow-up time of 2.0 s were then conducted.

Figure 1 shows the approximate major road one-way volume ranges for which a single-lane roundabout is recommended over two-way stop controlled and signalized intersections for both heavy vehicle percentage scenarios assumed in the analyses. For 10% heavy vehicles on both roads, the roundabout range is between 150 and 850 vph. When heavy vehicles on the major road are increased to 20%, this range shifts slightly to between 140 and 840 vph. Figure 2 shows the findings for all measures of effectiveness for 10% heavy vehicles on both roads. This figure indicates two sets of thresholds; one based solely on the effective intersection capacity and one based on a weighted average of all measures of effectiveness considered in this study. The recommended ranges shown in Figure 1 are those based on the weighted averages of the measures of effectiveness.

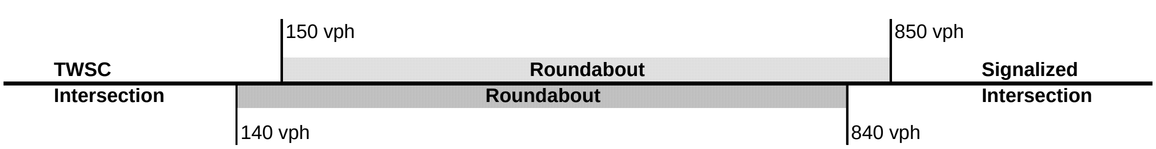
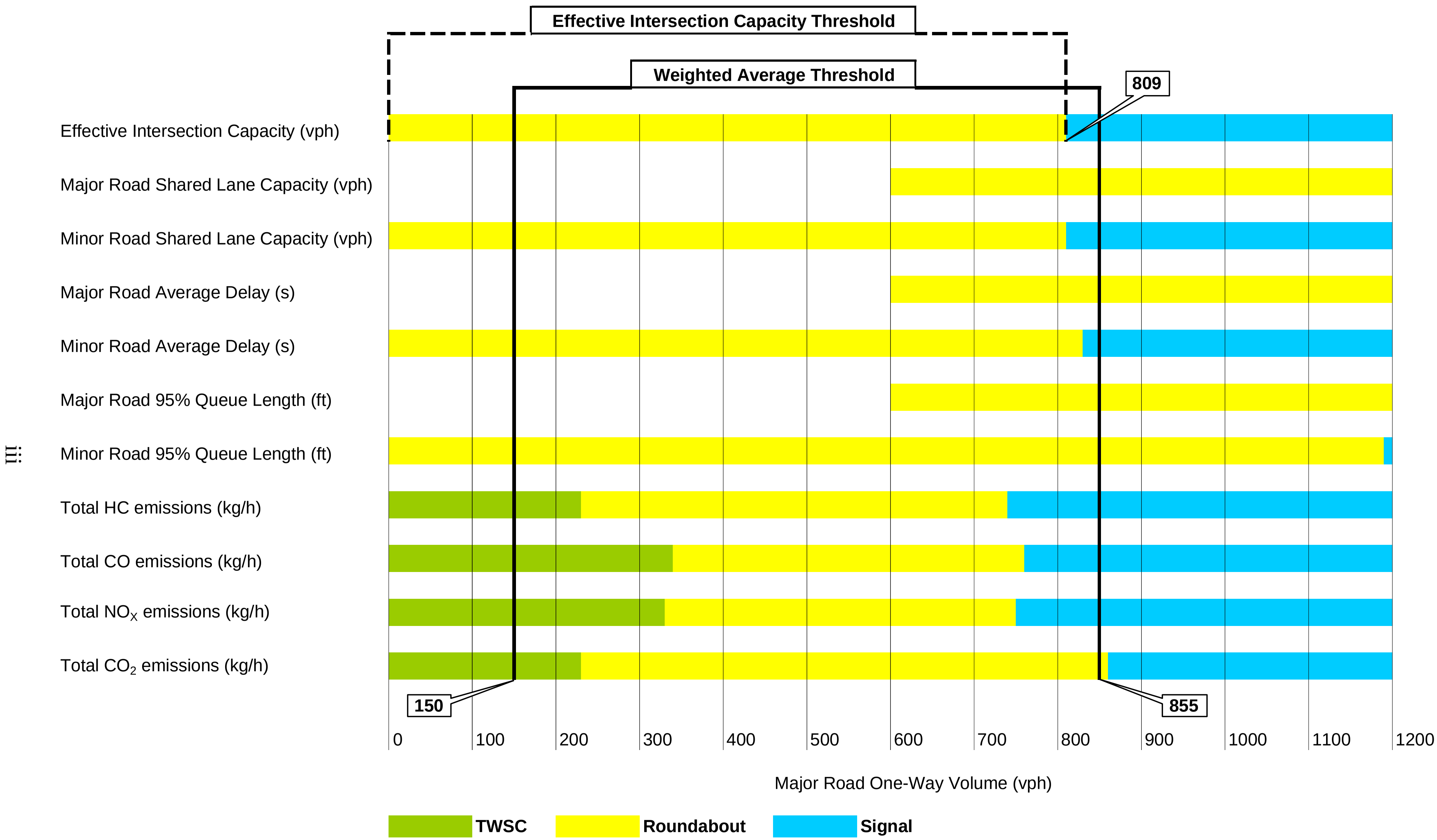


FIGURE 1 Recommended Ranges of One-way Major Road Volumes for which the Construction of Roundabouts is Preferred to that of TWSC or Signalized Intersections.
10% Heavy Vehicles on Both Roads (top), 20% Heavy Vehicles on the Major Road and 10% Heavy Vehicles on the Minor Road (bottom)



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1.0 INTRODUCTION

Roundabouts are garnering a growing interest in the United States partially because of their successful implementation abroad, particularly in Europe and Australia. When constructed where geometric and traffic conditions are appropriate, they can potentially provide advantages over conventional intersections in terms of capacity, delay, queue length, emissions, safety, and aesthetics. While similar to traffic circles, roundabouts are distinguished by several key features. Two fundamental elements are the yield-at-entry rule for entering vehicles and the deflection of approaches and exits so as to induce lower vehicle speeds. Other characteristics of roundabouts are the counter-clockwise movement of circulating vehicles, no parking permitted on the circulating and entry roadways, and the restriction of pedestrian movement to behind the yield sign on the legs of the roundabout.

In most typical cases, delay is generally evenly distributed among all approaches to a roundabout, in contrast to cross-intersections, where minor road vehicles experience significantly greater delay than major road vehicles. The yield control on roundabout approaches does not require that all vehicles come to a complete stop and therefore eliminates unnecessary stops made at traffic signals and stop signs when there are no conflicting vehicles. Additionally, minor road vehicles receive an "equal opportunity" to enter the intersection and so delays are typically reduced. This more efficient operation generally results in increased capacity and decreased overall delay.

Studies have shown that well-designed roundabouts typically reduce the frequency and severity of crashes occurring at intersections. Because of the overall improvement in safety associated with the use of roundabouts, they are often used as a means of traffic calming. In addition to lower vehicle speeds when compared to a cross-intersection, a roundabout reduces the number of potential vehicle-vehicle conflict points from 32 in a four-leg cross-intersection to 8. Both injury rates and the severity of crashes are reduced in roundabouts as a result of lower speeds and the elimination of right angle and left-turn head-on collisions, which are common at conventional intersections and are typically the most severe accidents.

It is often reported that roundabouts offer additional advantages over other intersection types. Maintenance costs for a roundabout are much less that those for traffic signals, as no major work is typically needed once a roundabout is constructed. Additionally, the center island of a roundabout provides space for landscaping and other aesthetic enhancements, which is appreciated by the local community and users of the roundabout.

This study will investigate the traffic conditions that may support the construction of roundabouts and will identify conditions under which roundabouts may be more appropriate than other types of intersection control, both signalized and unsignalized. The performance of each intersection type will be evaluated as traffic volumes and heavy vehicle percentages are varied along the intersecting roadways. Performance evaluations will be based upon several measures of effectiveness such as capacity, delay, queue length, and emission levels, which are provided by the SIDRA software package.

Preliminary analyses are conducted using default parameters as determined by the SIDRA software. It is known that driver behavior at roundabouts varies from country to country. In the United States in particular, driver behavior differs from that abroad, due to unfamiliarity throughout most of the country with roundabouts. Therefore, in order to localize the analysis results to Delaware driver behavior, it was decided to observe local roundabout sites to obtain gap acceptance parameters for use with the SIDRA software. However, due to the lack of roundabout sites in Delaware with adequate traffic volumes for a gap acceptance study, two Maryland locations were chosen for the study. These roundabouts are located in eastern Maryland, and, therefore, it is acceptable to assume that drivers are these sites are similar to Delaware drivers.

Based on the data collected in Maryland, an additional analysis using the SIDRA software was conducted to evaluate the measures of effectiveness for these local conditions. This analysis assumed a range of major road one-way volumes up to 1200 vehicles per hour and two heavy vehicle conditions on the major road of 10% and 20%.

The details of the analyses are presented in the body of this report along with the recommendations for ranges of major road one-way volumes that are most appropriate for construction of unsignalized intersections, roundabouts, and signalized intersections.

2.0 LITERATURE REVIEW

2.1 RECENT CAPACITY AND DELAY STUDIES: METHODOLOGY AND DATA COLLECTION

a. Troutbeck and Kako (1999)

Troutbeck and Kako (1999) developed a gap acceptance model for the merging process at congested unsignalized intersections. Unlike traditional gap acceptance models, which typically assume absolute priority of major stream vehicles over those of the minor stream, the proposed model assumes limited priority of major stream vehicles. Limited priority is a type of shared priority that is based on the assumption that major stream vehicles are slightly delayed in order to accommodate merging vehicles from the minor stream.

Field studies were conducted at three roundabouts located in Brisbane, Australia. All three roundabouts had two circulating lanes and two entry lanes, and normally operated under fairly high traffic volumes during peak hours. One hour of traffic data were collected with a videotape recorder at one entry of each roundabout during its morning or afternoon peak period. From each set of 1-hr data, two sets of 15-min data were extracted based upon traffic conditions at the entry approach being either "high saturation" or "low saturation." The frequency of events in which circulating vehicles were forced to slow down by entering vehicles was observed for each of these six data sets. The two-lane circulatory roadway was treated as a single lane and the times for circulating vehicles to cross two cross-sections, one upstream and one downstream of the entry, were recorded.

b. Polus and Shmueli (1997)

Polus and Shmueli (1997) developed an entry-capacity model for roundabouts that includes outside diameter and circulating flow as input parameters. Six small to medium-sized roundabouts in urban and suburban areas of Israel were included in this study. A separate regression model was developed for each roundabout studied because it was believed that the geometric characteristics of each site significantly affect its capacity. A general form of an exponential regression equation could then be developed. Results from the developed model were compared with those obtained from Australian and German models.

Flow and geometric data were collected from the six study sites. The capacity of each entry was defined as the maximum number of vehicles that can enter the roundabout in 1 hour under continuous queue conditions. Data were collected continuously by video camera, which were later assembled in 1-min intervals so that the flow encountered by each vehicle as it entered the roundabout could be determined. Data were collected only when a queue was formed at an entry, an indicator of a demand to enter the roundabout.

c. Polus and Shmueli (1999)

In a follow-up, Polus and Shmueli (1999) further examined and evaluated the capacity model previously developed in their 1997 study. In addition, the study estimated a gap size above which gaps are not relevant to the gap acceptance process and evaluated the gap acceptance behavior of drivers entering roundabouts as their waiting time on the approach leg increased.

Two relatively busy urban roundabouts in Israel were videotaped and data consisting of gaps, waiting times on the approach road, and circulating and entry volumes were collected.

d. Al-Masaeid and Faddah (1997)

In Jordan, Al-Masaeid and Faddah (1997) developed an empirical model for estimating entry capacity as a function of circulating traffic and geometric characteristics. Ten roundabouts located throughout Jordan were studied. Regression analysis was used to develop the entry-capacity model and its performance was then compared with results of German, Danish, and French capacity models.

The study sites experienced light pedestrian traffic despite their urban locations; no interference from pedestrians was encountered during data collection. Entry capacity was defined as the maximum traffic entering a roundabout during times of saturated demand. Circulating traffic flow and entry capacity data were collected manually for each roundabout entry at 1-min intervals during periods of continuous and stable queuing. Geometric variables were obtained through field measurements.

e. Al-Masaeid (1999)

Al-Masaeid (1999) used a logit analysis to develop models for estimating critical gap and move-up time at roundabouts. The first model predicts the probability that a random driver entering a roundabout will accept a given gap in the circulating stream based on geometric and gap characteristics. The second model estimates move-up time based on roundabout geometry and circulating traffic characteristics. Results from these models were incorporated into the Australian and German gap theoretical models to determine which of the two theoretical models is more appropriate for use in Jordan.

Gap is defined as the time interval between two circulating vehicles regardless of whether or not they use the same circulating lane. Total delay is calculated as the sum of queue delay and service time, where queue delay is the time from when a vehicle arrives at the end of the queue until it reaches the yield line. Service time is the time between a vehicle's arrival at and departure from the yield line. Move-up time is defined as the headway between two queued vehicles using the same gap in circulating traffic, measured as they enter the circulating flow.

The study sites consisted of ten roundabouts located throughout Jordan that operated at capacity and experienced considerable delay during peak periods. Data were collected for twenty selected approaches among the ten sites. Each roundabout had four or five approaches and each approach consisted of two to four entry lanes. Entry lanes were classified as dominant or subdominant based upon flow characteristics, where that with the greatest flow at each approach was classified as the dominant lane. Data were collected separately for each lane type using manual field techniques. Three data sets were collected. The first was used to establish critical gap models and consisted of geometric data, total delay times, and length of gaps rejected or accepted by entering drivers. The second was used to establish a move-up time model and consisted of traffic flow and average move-up time at 1-min intervals. The third data set was used to determine the most appropriate gap-acceptance model for Jordan roundabouts and consisted of entry capacity, circulating traffic flow, and total delay at 1-min intervals during peak and off-peak periods at one chosen roundabout.

f. Hagring (2000)

Hagring (2000) proposed a new capacity model for two-lane roundabouts based on previous studies (Hagring 1996, 1998) at Swedish roundabouts on the effects of origin-destination (OD) flows. The developed model was tested on two synthetic data sets and compared with another OD model proposed by Akçelik et al. (1996) and Akçelik (1997).

The previous work by Hagring studied critical gap differences between the inner and outer entry lanes at two-lane roundabout approaches. A simplified model was developed relating critical gap to the length and width of the weaving section between adjacent approaches. The capacity model presented and evaluated in the current study was first developed in these older studies.

g. Flannery, Elefteriadou, Koza, and McFadden (1998)

Flannery et al. (1998) studied five single-lane roundabouts before and after their installations to assess any changes in safety and operational performance. These intersections were located in Florida and Maryland and were stop-controlled prior to being converted to roundabouts. Two studies of these locations were conducted to determine the performance of the new roundabouts.

The first study consisted of collecting and comparing the average delay for vehicles before and after the sites were converted to roundabouts. Control delay was determined for entering vehicles by videotaping each site during its peak period. This study defined control delay as the time spent in queue, the move-up time in the queue, and the service time at the head of the queue. Field measurements of control delay at the sites before roundabouts were installed were not available. This delay was estimated for each approach using the procedure outlined in the 1997 update to Chapter 10 of the Highway Capacity Manual. The second study compared the field-measured delay with that predicted by the software SIDRA.

h. Flannery, Kharoufeh, Gautam, and Elefteriadou (2000)

Flannery et al. (2000) developed equations estimating the mean and variance of service time for a vehicle in the first position at an entry of a single-lane roundabout. With these estimates, the Pollaczek-Khintchine formula and Little's law may then be used to estimate the average number of queued vehicles and the average total waiting time per vehicle, respectively. Service time is defined as the time spent in the first position of the queue prior to entering the circulating stream and includes the time spent waiting for an acceptable gap in the circulating stream, travel time to enter the circulating stream, and the headway for the subsequent circulating vehicle. The total waiting time consists of the service time and the random time spent in the queue prior to assuming the first position. The model was developed using queuing theory, which is applicable under steady state conditions and assumes an M/G/1 queuing regime. The M/G/1 queuing regime assumes that the arrival process is Poisson and therefore the time between successive arrivals is exponential, or Markovian; the service times have a general distribution function; and there is a single server.

Driver performance and operational data were collected from six single-lane roundabouts located in Maryland and Florida. Cameras placed over the circulating roadway and each entry to the roundabout collected data for two hours during morning and evening peak periods.

i. Al-Omari, Al-Masaeid, and Al-Shawabkah (2004)

Al-Omari et al. (2004) developed a model for estimating roundabout delay as a function of traffic and geometric factors.

A total of twenty hours of field traffic and geometric data were collected from fourteen roundabouts located throughout Jordan. Data were collected on sunny days from locations with good pavement conditions and during times when there were no policemen in the area. It was not possible to collect data during congested conditions because traffic police control roundabouts at these times. Circulating volume, entry volume, and entry delay were measured during peak and non-peak periods using video cameras. Geometric design elements such as entry width and roundabout diameter were obtained through field measurements.

j. Várhelyi (2002)

Várhelyi (2002) investigated the effects of small roundabouts on emissions and fuel consumption at twenty-one locations in Växjö, Sweden. Of these sites, one was signalized and all others were yield-regulated prior to being replaced by small roundabouts. Traffic counts were conducted before and after the installation of the roundabouts for one workday at each site, between 7:30 AM and 5:00 PM. In order to register driving patterns, a "car-following" method was used in which randomly selected cars were followed with an instrument-equipped car that attempted to imitate the followed car's driving pattern as closely as possible.

Traffic counts were used to determine if volumes changed as a result of the new roundabouts and to determine the volume of emissions and fuel consumption per day at each site. Based on emission and fuel consumption factors developed by the Swedish car-testing institute *AB Svensk Bilprovning*, total emissions and fuel consumption for each followed car were calculated. Emissions factors were available for two types of passenger cars: A10-cars (petrol-driven without catalytic purifier) and L1-cars (petrol-driven with catalytic purifier). These factors are valid within limited intervals of speed and acceleration. In the rare event that speed or acceleration values were outside of the interval limits, the outermost values were used. The emissions for an average passenger car were determined using weighted factors of 0.7 for A10-cars and 0.3 for L1-cars, which reflect the percentage of Swedes driving each type of car in 1991. Heavy vehicles, which comprised about 7 percent of city traffic, were not considered in this study, as the emissions factors were applicable only to petrol-driven passenger cars.

k. Mandavilli, Russell, and Rys (2003)

Mandavilli et al. (2003) studied the impact of modern roundabouts on reducing vehicular emissions at intersections. The study sites consisted of six sites, experiencing different traffic conditions, at which single-lane modern roundabouts replaced stop-controlled intersections. Five of the sites were located in Kansas and one was located in Nevada. Prior to installation of the roundabouts, four of these sites were under two-way stop control and two sites were under all-way stop control.

Data collection consisted of two phases. The first phase consisted of videotaping intersection traffic movements for two six-hour sessions from 7:00 AM to 1:00 PM and from 1:00 PM to 7:00 PM on normal weekdays before and after the installation of the roundabouts. This study defines a normal weekday as a day having no inclement weather or other external factors, such as nearby events that would affect the traffic flow to the intersection. The second phase of data collection consisted of reviewing the videotapes to obtain AM and PM traffic counts for the before and after conditions. Data were recorded in 15-minute periods and the hourly data were then analyzed with the software aaSIDRA, version 2.0. All measures of effectiveness obtained from the SIDRA analysis were statistically compared with the software Minitab 13 using standard statistical procedures.

l. Russell, Mandavilli, and Rys (2005)

In another study of Kansas roundabouts, Russell et al. (2005) compared the operational performance of 11 intersections in 5 cites before and after their conversions to modern roundabouts. The study followed the same methodology as the earlier study by Mandavilli et al. (2004). The SIDRA software was used to provide measures of effectiveness so as to compare the intersection types. The MOEs chosen for this study are the 95th percentile queue length, degree of saturation, average intersection delay, maximum approach delay, proportion of vehicles stopped, and maximum proportion of vehicles stopped. Prior to the SIDRA analysis, statistical tests were performed to determined if the before and after traffic volumes were statistically similar.

2.2 CAPACITY AND DELAY MODELS

The following sections include capacity and delay models developed and used by government agencies as well as those developed in the studies discussed above.

2.2.1 CAPACITY MODELS

a. United Kingdom

Kimber (1980) conducted studies in the UK and developed an empirical linear regression based on a large number of observations at roundabouts operating at-capacity. This equation directly relates capacity to roundabout geometry.

$$Q_e = k(F - f_c Q_c) \tag{2.1}$$

Where:

Q_e = entry capacity, vph;
Q_c = circulating flow, pce/h; and
k, F, f_c = constants derived from the geometry of the roundabout.

b. Australia

Troutbeck (1993) conducted studies for the Australian Road Research Board and developed an analytical equation based on gap acceptance characteristics observed and measured at roundabouts operating below capacity. Critical gap and follow-up times are related to roundabout geometry and capacity is then determined using the following equation.

$$Q_e = \frac{\alpha Q_c e^{-\lambda(t_c - t_m)}}{1 - e^{-\lambda t_f}} \tag{2.2}$$

Where:

Q_e = entry capacity, vph;
Q_c = circulating flow, vph;
α = proportion of non-bunched (free) vehicles in the circulating streams;
λ = model parameter;
t_c = critical gap, s;
t_f = minimum headway in circulating streams, s; and
t_m = follow-up time, s.

c. Germany

Stuwe (1992) studied 11 German roundabouts and developed the following exponential regression equation to estimate roundabout entry capacity.

$$C = Ae^{\left(-\frac{BV_C}{10,000}\right)} \qquad (2.3)$$

Where:

C = entry capacity, vph;
V_c = circulating flow, vph; and
A, B = parameters dependent on the number of circulating and entry lanes.

d. Switzerland

Bovy (1991) developed a linear model based on studies of roundabouts in Switzerland.

$$C = 1500 - (8/9)V_g \qquad (2.4)$$

Where:

C = entry capacity, vph; and
V_g = impeding flow vph, determined with the equation below.

$$V_g = \beta V_c + \alpha V_s \qquad (2.5)$$

Where:

V_c = circulating flow, vph;
V_s = exiting flow, vph; and
α, β = parameters depending on geometry and number of circulating lanes.

e. France

The French government organization Centre d'Etudes des Transports Urbains (CETUR), now known as CERTU, developed a linear model for urban roundabout capacity in 1988.

$$C = 1500 - (5/6)V_g \tag{2.6}$$

Where:

C = entry capacity, vph; and
V_g = impeding flow, vph, determined with the equation below.

$$V_g = V_c + \alpha V_s \tag{2.7}$$

Where:

V_c = circulating flow, vph;
V_s = exiting flow, vph; and
α = parameter dependent on splitter island width.

SETRA, the French national design organization for rural highways, developed a linear equation for the capacity of rural roundabouts in 1987.

$$C = (1330 - 0.7V_g)(1 + 0.1[l_e - 3.5]) \tag{2.8}$$

Where:

C = entry capacity, vph;
V_g = impeding flow, vph, dependent on circulating and exiting flows and geometry; and
l_e = entry width, m.

f. United States

The Highway Capacity Manual 2000 (HCM) presents methodology for estimating roundabout capacity based on gap acceptance. The capacity model presented is applicable only to single-lane roundabouts and if the circulating volume is less than 1,200 vph. Insufficient experience in the U.S. precludes the HCM from containing guidelines for multiple-lane roundabouts. The HCM assumes that the gap acceptance characteristics of drivers entering a roundabout to be similar to those of drivers making right turns at two-way stop-controlled intersections.

$$c_a = \frac{v_c e^{-v_c t_c / 3600}}{1 - e^{-v_c t_t / 3600}} \qquad (2.9)$$

Where:

c_a = approach capacity, vph;
v_c = conflicting circulating traffic, vph;
t_c = critical gap, s; and
t_t = follow-up time, s.

The HCM 2000 recommends ranges for critical gap and follow-up times, which are presented in Table 2.1.

TABLE 2.1 HCM Critical Gap and Follow-up Times (HCM 2000, Exhibit 17-37)

	Critical gap (s)	*Follow-up time (s)*
Upper bound	4.1	2.6
Lower bound	4.6	3.1

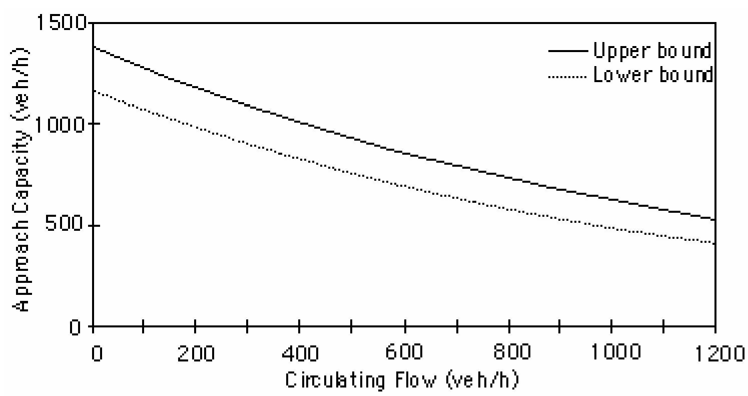


FIGURE 2.1 Roundabout Approach Capacity (HCM 2000, Exhibit 17-38)

The U.S. Federal Highway Administration (FHWA) publication, *Roundabouts: An Informational Guide* (2000) presents a more comprehensive discussion of roundabout performance analysis than the HCM 2000. This document differentiates roundabouts based upon size and environment and provides capacity models for urban compact roundabouts, typical single-lane roundabouts, and typical double-lane roundabouts.

Urban compact roundabouts have nearly perpendicular single-lane approach legs and inscribed circle diameters in the range of 25 to 30 m (82 to 98.5 ft). The capacity model for urban compact roundabouts is based on the capacity curves developed by Brilon, Wu, and Bondzio (1997) for German roundabouts with single-lane entries and a single-lane circulatory roadway.

$$Q_e = 1218 - 0.74 Q_c \tag{2.10}$$

Where:

Q_e = entry capacity, vph; and
Q_c = circulating flow, vph.

The British equations developed by Kimber (1980) form the basis for the capacity models derived for typical single-lane and double-lane roundabouts. The indicated assumptions of geometric parameters were chosen so as to simplify the equations.

Single-lane roundabouts:

$$Q_e = Min\{(1212 - 0.5447 Q_c), (1800 - Q_c)\} \tag{2.11}$$

Assuming: D = 40 m, r = 20 m, ϕ = 30°, v = 4 m, e = 4 m, l' = 40 m.

Double-lane roundabouts:

$$Q_e = 2424 - 0.7159 Q_c \tag{2.12}$$

Assuming: D = 55 m, r = 20 m, ϕ = 30°, v = 8 m, e = 8 m, l' = 40 m.

Where:

Q_e = entry capacity, vph;
Q_c = circulating flow, vph;
D = inscribed circle diameter, m;
r = entry radius, m;
ϕ = entry angle, degrees;
v = approach half width, m;

e = entry width, m; and
l' = effective flare length, m.

When capacity requirements are met at double-lane roundabouts with the gradual widening of the approaches (flaring), the capacity of each entry lane, q_{max}, is estimated with the equation below. Equation 2.13 is based on Wu's (1997) studies on the effect of short lanes on entry capacity for two-lane roundabouts.

$$q_{\max} = \frac{2q}{x^{n} \sqrt[n+1]{2}} = \frac{q_{\max 2}}{\sqrt[n+1]{2}} \qquad (2.13)$$

Where:

q = flow in each lane, pce/h (assumed to be equal in both lanes);
x = degree of saturation;
n = length of queue space, vehicles; and
$q_{max\ 2}$ = capacity of entry at a double-lane roundabout, pce/h.

Figure 2.2 shows a comparison of entry capacities based on the equation provided by the FHWA *Roundabouts: An Informational Guide.*

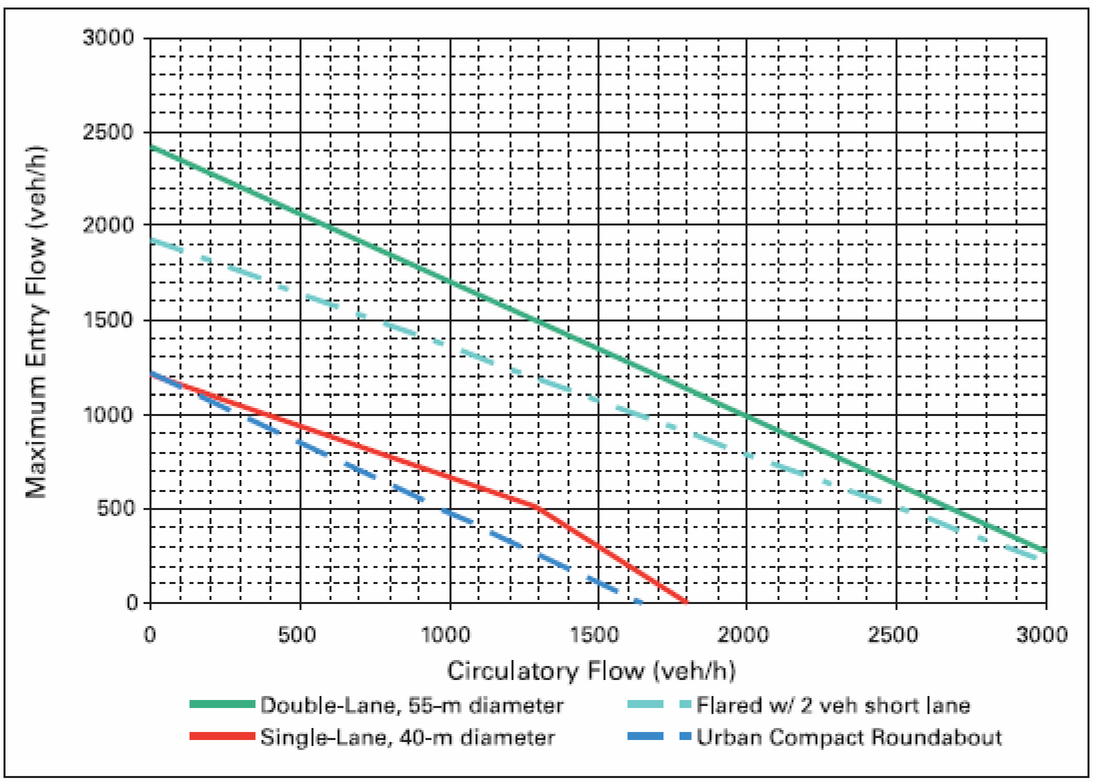


FIGURE 2.2 Entry Capacities for Single and Double-lane Roundabouts (FHWA, 2000)

g. Polus and Shmueli (1997)

Polus and Shmueli developed an exponential regression model based on their studies of six small to medium-sized roundabouts in Israel. The model yields an exponential decrease in entry capacity with an increase in circulating flow and a significant increase in entry capacity with an increase in outside diameter.

$$C = 394D^{0.31}e^{-(0.00095Vc)} \tag{2.14}$$

Where:

C = entry capacity, vph;
D = outside diameter, m; and
Vc = circulating flow, vph.

Figure 2.3 presents circulating flow versus entry capacity for the six study sites.

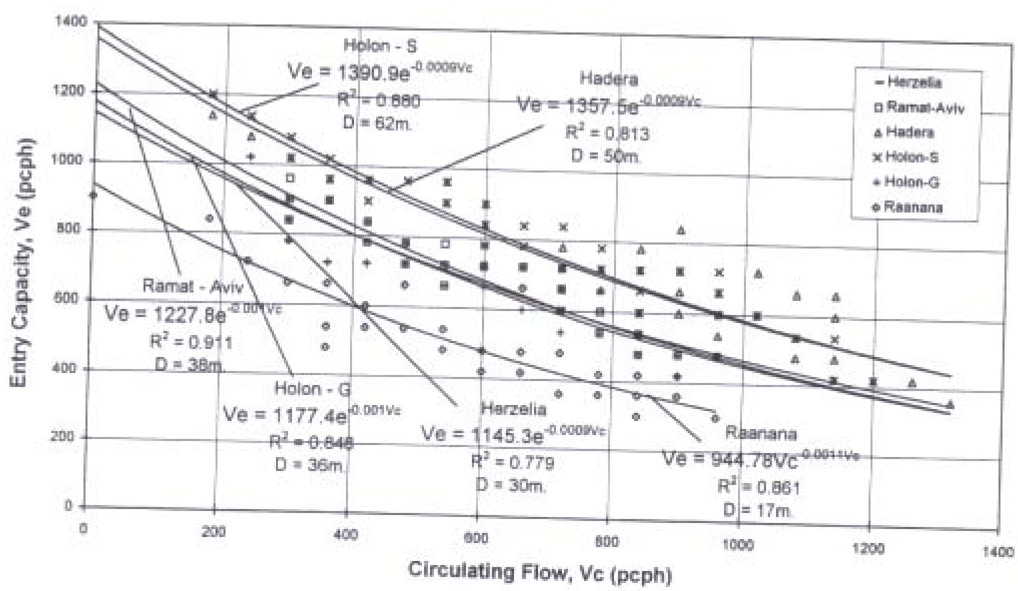


FIGURE 2.3 Models for Entry Capacities versus Circulating Flows for Study Sites (Polus and Shmueli, 1997)

Shiftan, Polus, and Shmueli-Lazar (2003) used a binary logit model to estimate the effect of waiting time at a roundabout entry on the likelihood of accepting different gaps. The model, presented as Figure 2.4, shows that critical gap decreases as waiting time increases. For example, after a waiting time of 10 seconds, the critical gap was about 4 seconds whereas after a waiting time of 80 seconds, the critical gap was about 3 seconds.

In capacity models for which the critical gap is an input, such a reduction would significantly increase entry capacity.

FIGURE 2.4 Critical Gap as a Function of Waiting Time (Shiftan et al., 2003)

Polus, Shmueli-Lazar, and Livneh (2003) found that an increase in waiting time results in a decrease in critical gap. The logistic model developed for the decrease in critical gap has an s-shaped curve between the maximum and minimum critical gap values and is presented as Figure 2.5. The authors also calibrated an exponential capacity model that includes critical gap as an input.

$$C = 394D^{0.31}e^{-0.00023t_{cr}V_c} \tag{2.15}$$

Where:

C = entry capacity, vph;
D = outside diameter, m;
t_{cr} = critical gap, s; and
V_c = circulating volume, vph.

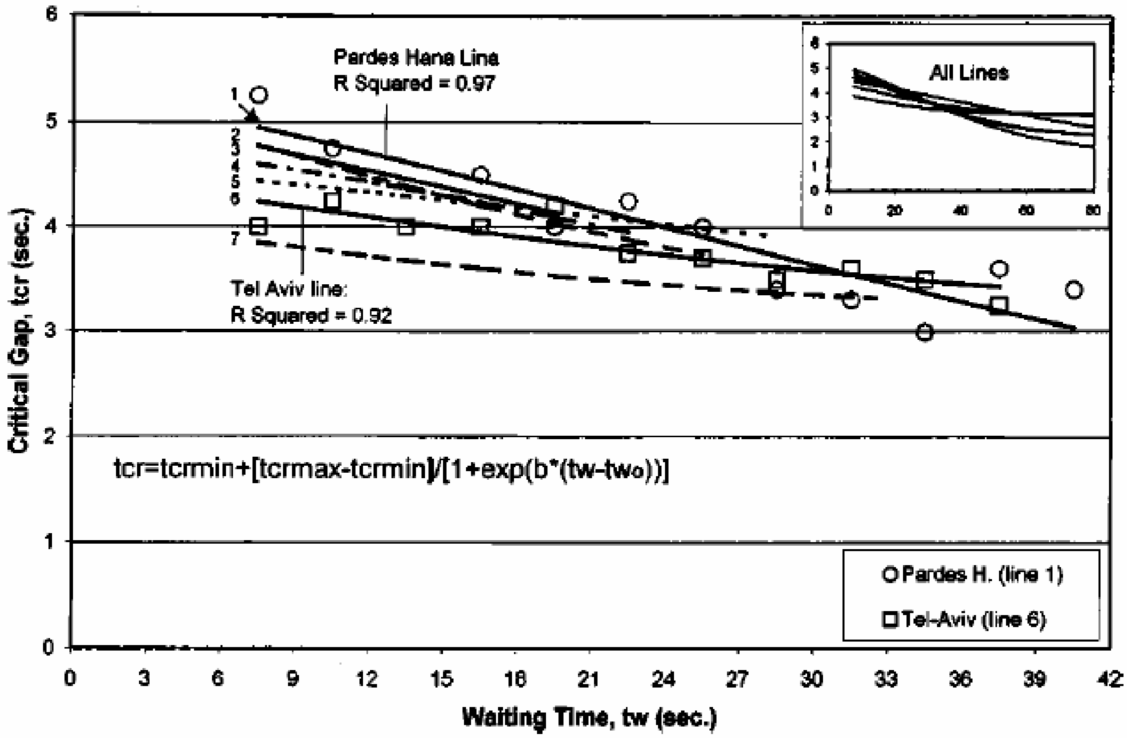


FIGURE 2.5 Calibration of S-curves for Critical Gaps Versus Waiting Times (insert shows entire range of curves) (Polus et al., 2003)

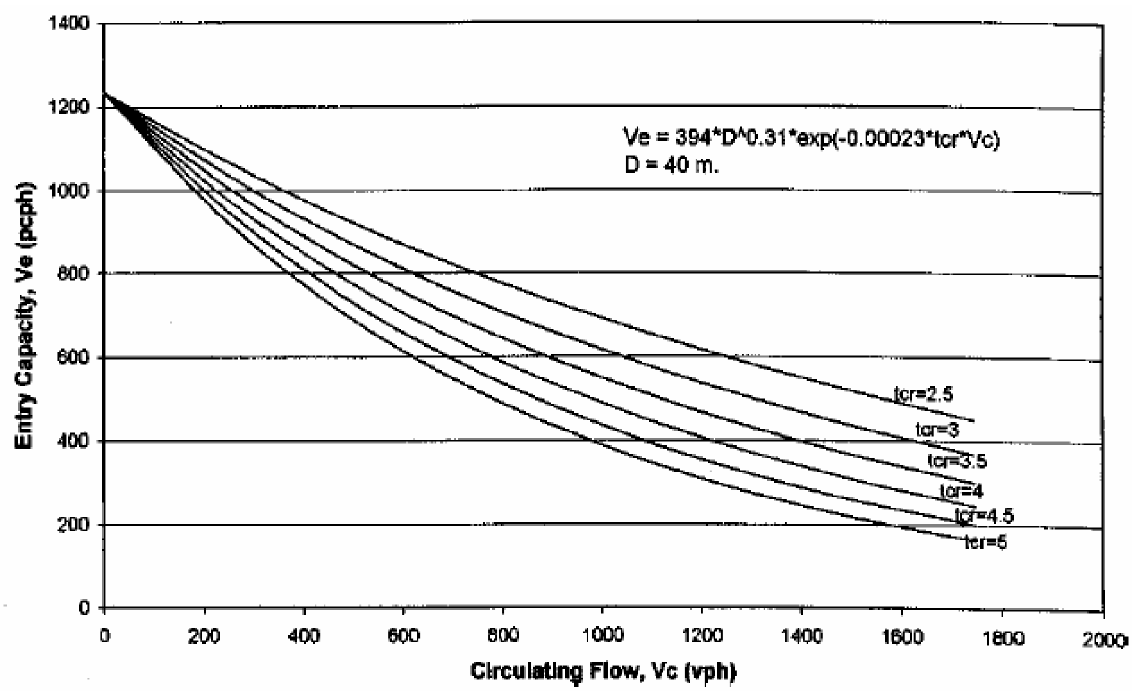


FIGURE 2.6 Model for Entry Capacity for Various Critical Gaps (Polus et al., 2003)

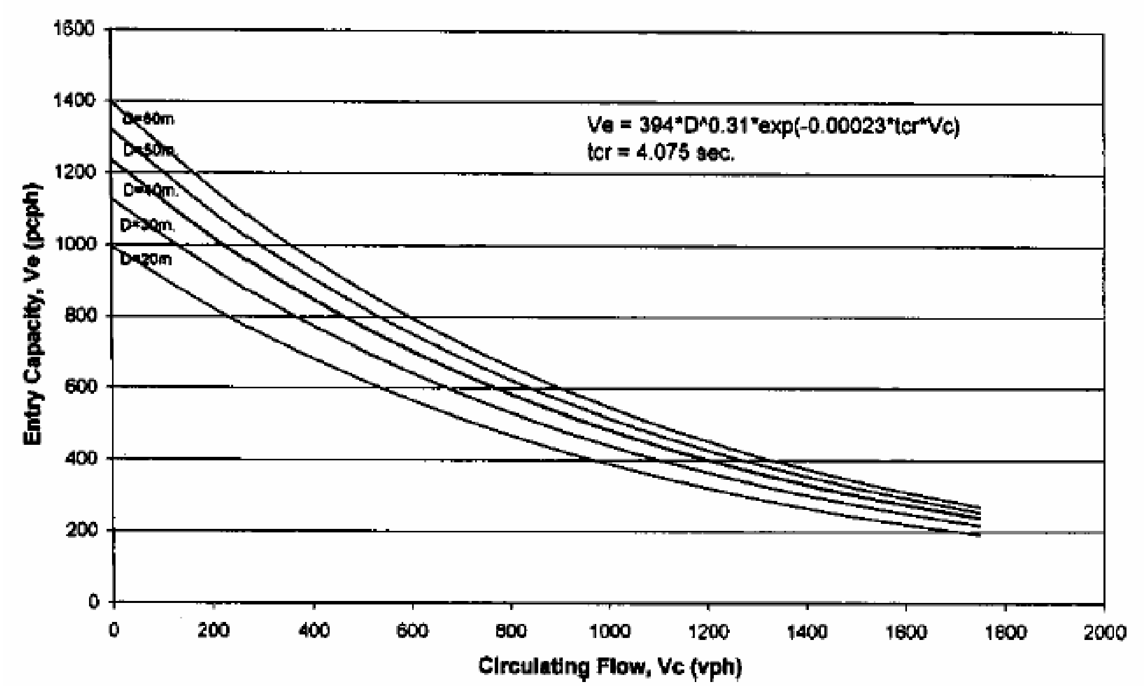


FIGURE 2.7 Model for Entry Capacity for Various Outside Diameters
(Polus et al., 2003)

h. Polus and Shmueli (1999)

Polus and Shmueli (1999) found that the capacities predicted by the model were generally consistent with the observed capacities of the two roundabouts. At one site, however, it was observed that a 6.5% upgrade on the approach resulted in an increase in gap acceptance values. The following model was also developed for determining the maximum relevant gap at roundabouts.

$$G_t = \frac{\pi R}{S} = k\left(\frac{R}{e+f}\right)^{1/2} \tag{2.16}$$

Where:

- G_t = threshold gap, s;
- R = radius of the vehicle path around the central island, m;
- S = operating speed around the circle, m/s;
- e = superelevation of the vehicle path around the central island, m/m;
- f = side friction coefficient; and
- k = coefficient representing local conditions, s/m.

The critical gaps were found to be 4.1 and 4.2 s. It was also found that the critical gap decreased as the average waiting time on the entry leg exceeded 9 seconds. The authors calibrated the following model for predicting the critical gap when considerable waiting times (longer than 9 seconds) exist on an entry leg. Figure 2.8 shows the calibrated model and it can be seen that for waiting times approaching 25 seconds, the critical gap is decreased to near or below 4 seconds.

$$T_{CW} = 5.757e^{-0.0142WT} \tag{2.17}$$

Where:

T_{CW} = critical gap at the roundabout, s; and
WT = waiting time on the entry leg, s.

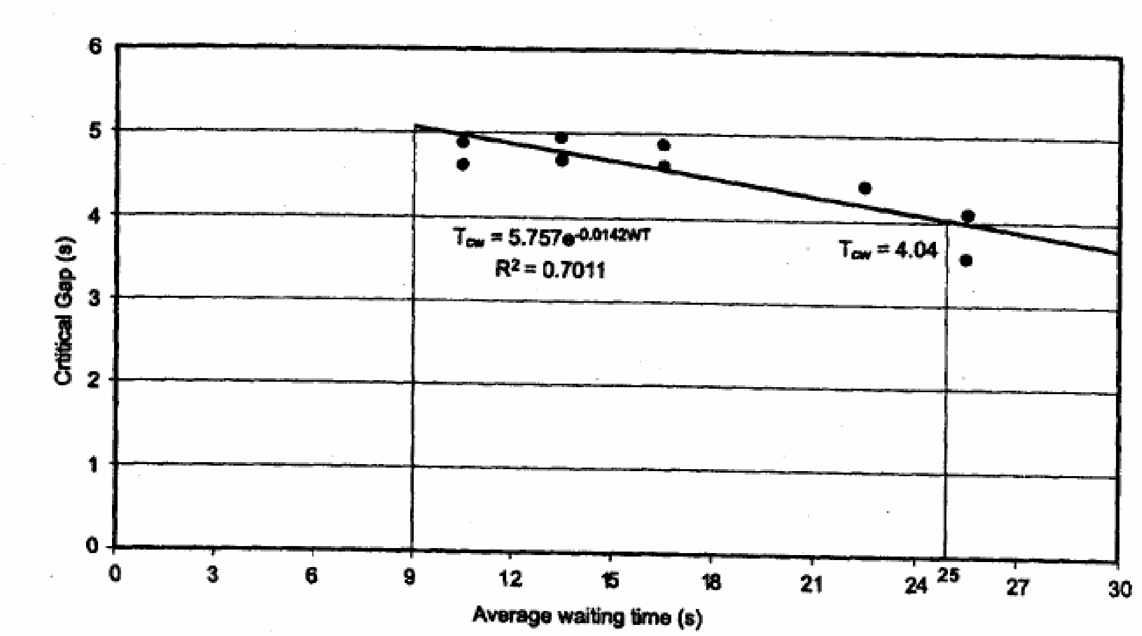


FIGURE 2.8 Relationship Between Critical Gap and Average Waiting Time
(Polus and Shmueli, 1999)

i. Al-Masaeid and Faddah (1997)

The following general equation was found to best fit the capacity data after a multivariate regression analysis was conducted

$$q_e = 168.2D^{0.312}S^{0.219}e^{0.071EW+0.019RW}e^{-5.602q_C/10,000} \tag{2.18}$$

Where:

q_e = entry capacity, vph;
D = central island diameter, m;

S = distance between entry and near-side exit, m;
EW = entry width, m;
RW = circulating roadway width, m; and
q_c = circulating traffic flow, vph.

Entry width and central island diameter were found to have the greatest effect on the estimated entrance capacity while the circulating roadway width was found to have the least relative effect. Figure 2.9 presents a scatterplot of circulating flow versus entry capacity for a selected entry.

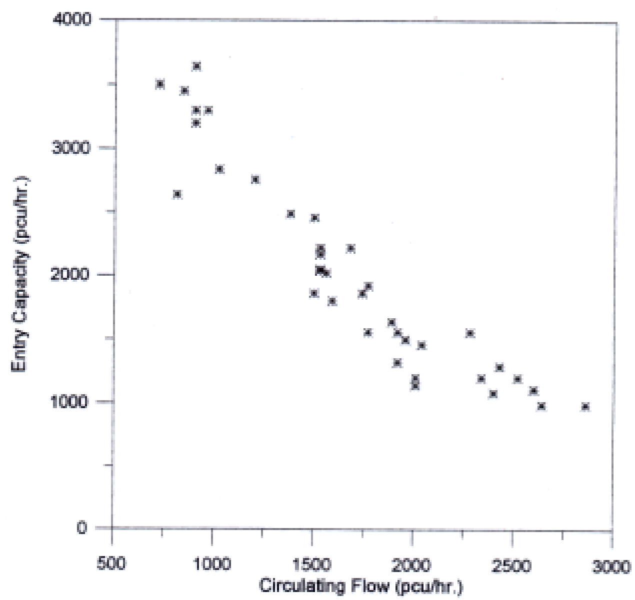


FIGURE 2.9 Entry Capacity Versus Circulating Flow for a Selected Roundabout Entry (Al-Masaeid and Faddah, 1997)

j. Al-Masaeid (1999)

Al-Masaeid found that gap length and total delay are significant predictors of gap-acceptance behavior. For a driver entering from a dominant or subdominant lane, the probability that a given gap will be accepted is determined with Equations 2.19 and 2.20, respectively.

$$P(accept) = \frac{1}{1 - \exp(22.466 - 3.628G - 0.110D - 0.079TD)} \tag{2.19}$$

$$P(accept) = \frac{1}{1 - \exp(19.929 - 2.660G - 0.096D - 0.054TD)} \quad (2.20)$$

Where:

G = length of gap, s;
D = roundabout diameter, m; and
TD = total delay, s.

Using multiple regression analysis, Equations 2.21 and 2.22 were obtained to estimate move-up time.

$$\beta_d = 3.31 - 0.013D - 0.004S - 0.36F \quad (2.21)$$

$$\beta_s = 3.86 - 0.013D - 0.003S - 0.38F \quad (2.22)$$

Where:

β_d = move-up time for drivers in dominant lane, s;
β_s = move-up time for drivers in subdominant lane, s;
D = roundabout diameter, m;
S = distance between roundabout entry and near side next exit, m; and
F = average circulating traffic flow per lane, thousands passenger car units per hour.

Al-Masaeid found the effect of roundabout diameter on gap-acceptance behavior in the dominant and subdominant lanes to be approximately equal. However, vehicles in the dominant lane are more sensitive to gap length and total delay than are those in subdominant lanes. Figure 2.10 shows the effect of diameter on the critical gap for a dominant lane at a total delay of 10 s. For the same diameter and delay, vehicles in the dominant lane are willing to accept smaller gaps in the circulating stream and, therefore, dominant lanes have greater capacity. Al-Masaeid's finding that entry capacity increases as diameter increases is consistent with those of previous studies, namely, Troutbeck (1993), Al-Masaeid and Faddah (1997), and Polus and Shmueli (1997).

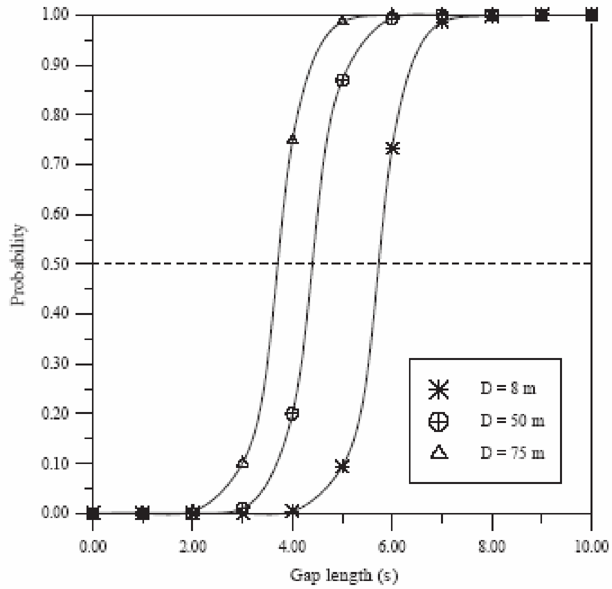


FIGURE 2.10 Effect of Diameter on Critical Gap of a Dominant Lane (total delay 10 s) (Al-Masaeid, 1999)

Al-Masaeid identified total delay as an influencing factor in the probability that an entering driver accepts a gap. This is significant in that total delay had not been considered in modeling driver decision at roundabout entry prior to this study. Figure 2.11 shows that the entry capacity increases as delay increases. This is expected because critical gap and move-up time values decrease the longer drivers wait at the entry.

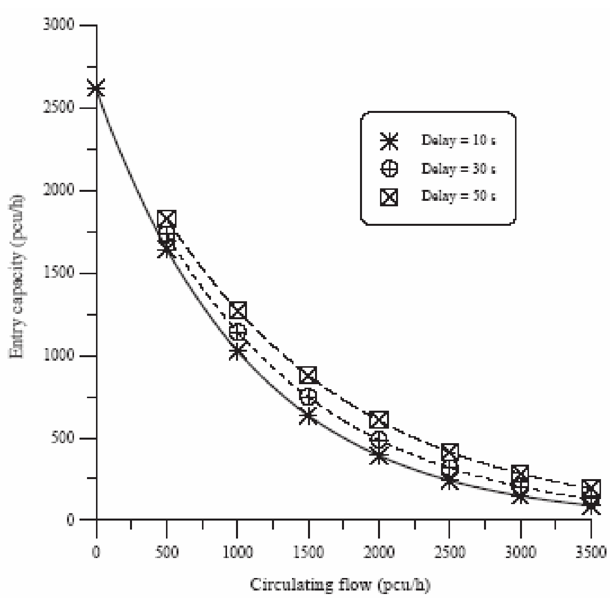


FIGURE 2.11 Estimated Entry Capacity Versus Circulating Flow at Different Total Delay Levels (Al-Masaeid, 1999)

k. Hagring (2000)

Hagring found that capacity is strongly influenced by the allocation of the circulating flow on the two circulating lanes. The maximum capacity is obtained when the circulating flow is equally distributed between the two lanes and the minimum when the flow is allocated to one lane only. This proposed model assumes an equal degree of saturation of the entering vehicles onto entrance lanes and is based on a generalized M3 distribution of headways in the circulating stream. Cowan's (1975) M3 model is a dichotomized headway model which assumes a proportion, α, of free vehicles and a proportion, 1-α, of bunched vehicles.

$$C = \Lambda \prod_i \frac{\alpha_i q_i}{\lambda_i} \frac{e^{-\Sigma_k \lambda_k T_k}}{e^{-\Lambda\Delta}(1 - e^{-\Sigma_m \lambda_m T_{0m}})} \qquad (2.23)$$

Where:

C = capacity;
$\Lambda = \sum_i \lambda_i$;
α = 0.910-1.545q = proportion of free vehicles in lane *i*;
q = flow in lane *i;*
λ = (qα)/(1-qΔ) = intensity for longer gaps in lane *i*;
T = critical gap, s;
Δ = minimum headway between vehicles, s; and
T_0 = follow-up time, s.

Hagring suggested two algorithms for use in this capacity model allocating the circulating flow between the two circulating lanes. The difference between the two is the treatment of left-turning vehicles from the opposite approach. While the first algorithm assumes that they choose the near or far circulating lane with equal probability, the second assumes that all vehicles choose the near circulating lane

Earlier studies by Hagring (1996, 1998) found that critical gap differs between the two entering lanes at a two-lane roundabout. The studies also found that right-turning vehicles in the outer entry lane had significantly smaller critical gaps than those of other turning movements at the same approach. Critical gaps were found to be very similar regardless of which of the two circulating lanes vehicles were entering and it was therefore concluded that both circulating streams impede entering vehicles. Hagring related critical gap to the size of the weaving area between two adjacent roundabout approaches with Equation 2.24.

$$T = 3.91 - 0.0278L + 0.121w + 0.592(N_L - 1) \tag{2.24}$$

Where:

T = critical gap;
L = length of weaving section;
w = width of weaving section; and
N_L = lane number (outer lane = 1, inner lane = 2).

Figures 2.12 and 2.13 show the estimations of capacity for one roundabout approach using two synthetic data sets.

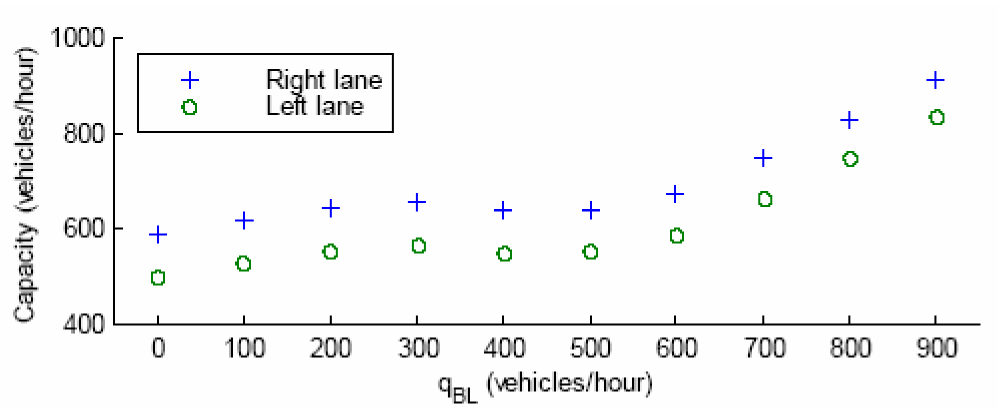


FIGURE 2.12 Estimated Capacity Versus Left-turn Flow of Opposite Approach, Data Set 1 (Hagring, 2000)

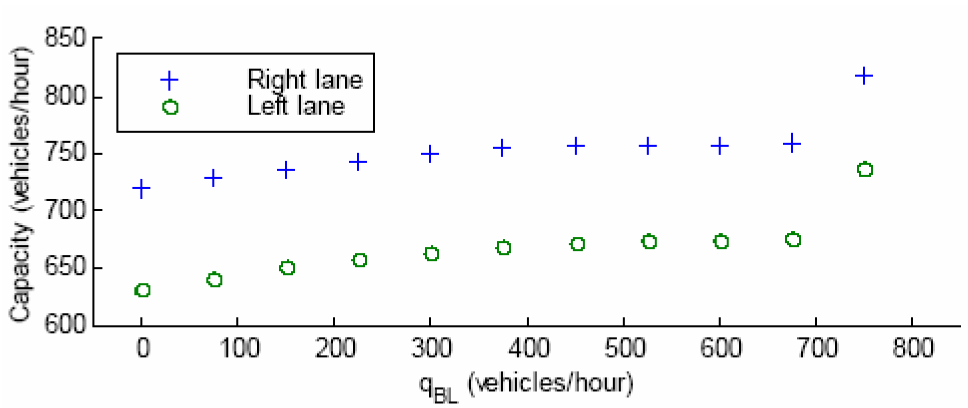


FIGURE 2.13 Estimated Capacity Versus Left-turn Flow of Opposite Approach, Data Set 2 (Hagring, 2000)

2.2.2 DELAY STUDIES AND MODELS

a. FHWA *Roundabouts: An Informational Guide*

The HCM 2000 does not provide guidance for estimating delay at roundabouts. The FHWA Roundabout Guide provides a modified version of the equation given by the HCM for control delay at two-way stop-controlled (TWSC) intersections as an estimate for roundabout control delay. Equation 2.23 omits the additional 5 s included in the HCM equation that accounts for deceleration to and acceleration from a complete stop at a TWSC intersection. This is because control delay at roundabouts may be less than 5 s, as vehicles are not required to come to a complete stop.

$$d = \frac{3600}{c_{m,x}} + 900T \times \left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{450T}} \right] \qquad (2.25)$$

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; and
T = analysis time period, h (T=0.25 for a 15-minute period).

Figure 2.14 presents control delay as a function of varying capacity and entering flow values. Note the exponential shape of the relationships.

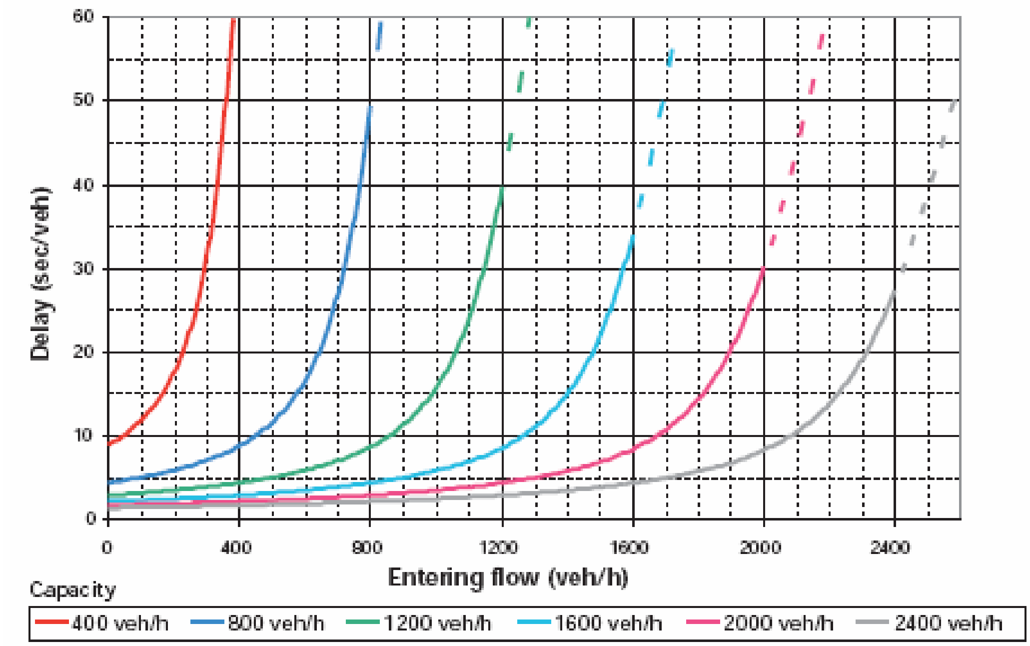


FIGURE 2.14 Control Delay as a Function of Capacity and Entering Flow (FHWA, 2000)

b. Flannery, Elefteriadou, Koza, and McFadden (1998)

As shown in Table 2.2, all but one site experienced a reduction in control delay after the roundabouts were installed. This is most likely due to the change in the spread of delay among the approaches; where once some approaches had delay while others had none, there is now a more uniform delay for all.

TABLE 2.2 Control Delay Before and After the Installation of Roundabout (Flannery et al., 1998)

Location	*Estimated avg. control delay in before period (stop-controlled) (s/veh)*	*Measured avg. control delay in after period (roundabout) (s/veh)*
Palm Beach Co., Florida	2.71	0.97
Lisbon, Maryland	10.36	1.62
Tallahassee, Florida	5.58	9.04
Ft. Walton Beach, Florida	163.52	3.36
Lothian, Maryland	9.62	4.15

c. Flannery, Kharoufeh, Gautam, and Elefteriadou (2000)

In order to develop a model that is not dependent on a specific headway distribution in the circulating stream, the queuing system was modeled as an M/G/1 queuing regime.

An M/G/1 queuing regime consists of a Poisson arrival process and hence exponential (or Markovian) time between successive arrivals (M), a general distribution of service times (G), and a single server (1). The queuing model assumes instantaneous service, i.e., zero time for vehicle passage into the circulating stream as well as infinite physical space for queue storage. Assuming the same gap acceptance criteria for all vehicles, the mean average accepted gap during each study period was used to estimate the mean and variance of service time.

Denoting headway times in the circulating stream as some general distribution function $F = (\cdot)$, the expected service time is given below.

$$E(T) = \frac{1}{\tau}\left\{\frac{1}{2}g^2 - \int_0^g tF(t)dt - (1 - f(g))^{-1}\left[g - \int_0^g F(t)dt \int_0^g tdF(t)\right]\right\} \qquad (2.26)$$

Where:

E(T) = expected service time, s;
g = mean acceptable gap size for drivers arriving at the approach, s; and
τ = mean time headway for circulating vehicles, s.

The variance of service time is given the following equation.

$$\begin{aligned} VAR(T) &= \frac{1}{\tau}\left(\frac{1}{3}g^3 + E(T_0)g^2 - \int_0^g \left(t^2 + 2tE(T_0)\right)\cdot F(t)dt\right) \\ &+ \frac{1}{\tau}\left(g - \int_0^g F(t)dt(1 - F(g))^{-1}\right)\cdot\left(\int_0^g t^2 dF(t) + 2E(T_0)\int_0^g tdF(t)\right) - [E(T)]^2 \end{aligned} \qquad (2.27)$$

where the last term is obtained by Equation 2.26 and $E(T_0) = (1 - F(g))^{-1}\int_0^g tdF(t)$.

The mean and variance of service time are then used with the Pollaczek-Khintchine formula to estimate the average number of queued vehicles *L* and Little's law to estimate the mean waiting time *W*.

$$L = \lambda E(T) + \frac{\lambda^2\left[(E(T))^2 + VAR(T)\right]}{2(1 - \lambda E(T))} \qquad (2.28)$$

Where:

L = average number of queued vehicles; and

λ = mean arrival rate to the queue.

$$W = \frac{L}{\lambda} \tag{2.29}$$

Where:

W = mean waiting time;
L = average number of queued vehicles (from Equation 2.22); and
λ = mean arrival rate to t he queue.

Table 2.3 shows a comparison between results obtained from the developed models and those observed in the field. The headway distribution in the circulating stream was modeled as lognormal, as this was found to best represent the observed conditions. It is important to note that the developed model facilitates the use of any headway distribution in the circulating stream.

TABLE 2.3 Deviation Between Field and Modeled Values (lognormal headway) (Flannery et al., 2000)

Statistic	*Mean service time (s)*	*Std. dev. of service time (s)*
Mean Absolute Deviation	1.2710	1.3727
Max. Absolute Deviation	6.9500	6.0784

d. Al-Omari, Al-Masaeid, and Al-Shawabkah (2004)

Using 15-minute time intervals, the empirical model shown as Equation 2.30 was developed (see Figure 2.15).

$$D_s = 0.0027V_s + 0.0056V_c - 0.1802D_i + 0.8048W_c - 0.3083W_e \tag{2.30}$$

Where:

D_s = stopped delay; sec/veh;
V_s = volume of vehicles in the subject entry, vph;
V_c = volume of vehicles in the circulating roadway, vph;
D_i = diameter of the roundabout island, m;
W_c = width of the circulating roadway, m; and
W_e = width of the subject approach entry, m.

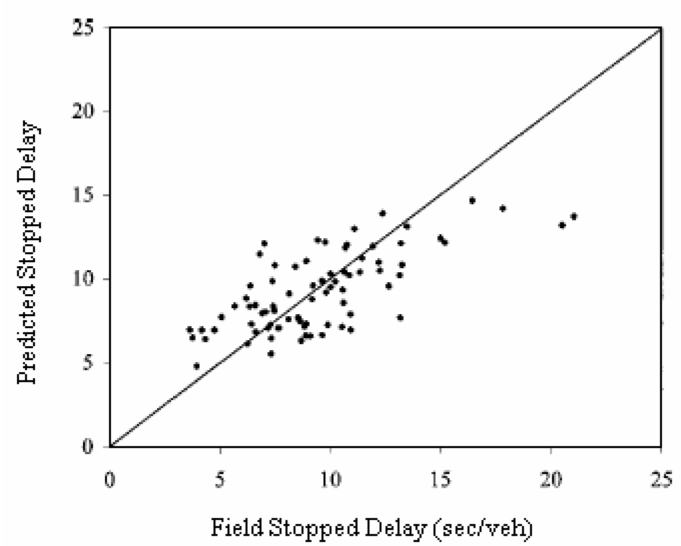


FIGURE 2.15 Proposed Model versus Field Stopped Delay (Al-Omari et al., 2004)

e. Russell, Mandavilli, and Rys (2005)

The study concluded that for the intersections studied, modern roundabouts operate more efficiently than the before traffic control. Following the conversion to modern roundabouts, the study found a 65% decrease in the average intersection delay, a 71% decrease in the maximum approach delay, a 44% decrease in the 95% queue length, a 53% decrease in the degree of saturation, a 52% decrease in proportion of vehicles stopped, and a 42% decrease in the maximum proportion of vehicles stopped. All decreases were found to be statistically significant. Since the delay, queue, and proportion of vehicles stopped were significantly reduced, the authors conclude that the operational efficiency of these intersections should therefore improve significantly.

2.2.3 RECENT CAPACITY AND DELAY STUDIES: COMPARISONS

a. Troutbeck and Kako (1999)

Figure 2.16 compares the entry capacity for a given two-lane roundabout using several methods. The AUSTROADS (Australian) method, based on the absolute priority system, produces a pronounced S-shaped curve and appears to significantly overestimate capacity compared with the empirical UK method, which is represented by the straight line. Another gap acceptance model, used in the SIDRA 5 software, appears to underestimate capacity at higher circulating flows and does not produce a linear relationship, as does the UK method. In contrast to both the AUSTROADS and SIDRA 5 models, the limited priority capacity equation yields an almost linear relationship that is very close to the UK relationship. The study concluded that the capacity prediction based on the limited priority system is reasonable and provides a theoretical basis for the empirical UK methods.

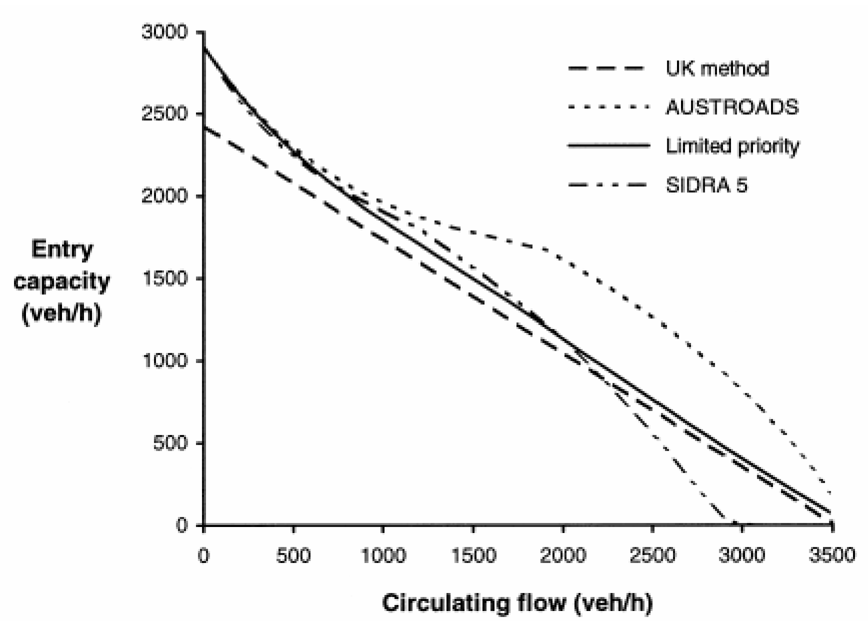


FIGURE 2.16 Entry Capacity for a Two-lane Roundabout With a 60-m Inscribed Diameter and a 4-m Entry Lane Width (Troutbeck and Kako, 1999)

b. Polus and Shmueli (1997)

Capacity estimates were obtained with the Australian, German, English, and Swiss models to compare against those from the developed model. The results from the proposed model show a good fit to the other models, as seen in Figure 2.17. A very close fit to the German model was obtained, though it is not dependent on the diameter. The Australian model yielded slightly higher entry capacities for moderate to low circulating flows and lower entry capacities for high circulating flows.

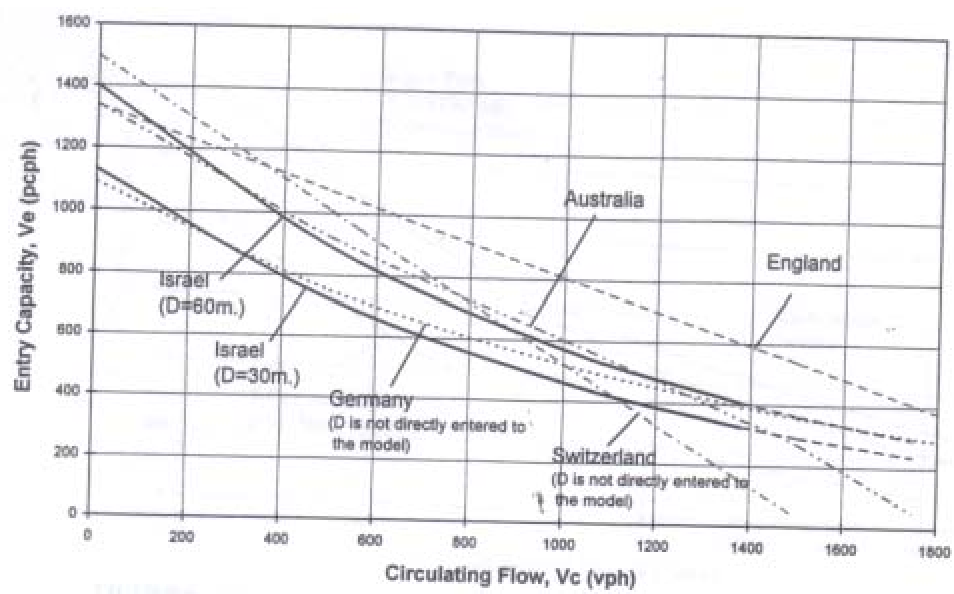


FIGURE 2.17 Entry Capacities Versus Circulating Flows for *D* Between 30 and 60 m (Polus and Shmueli, 1997)

c. Al-Masaeid and Faddah (1997)

Figure 2.18 shows the relationships between estimated entry capacity and circulating traffic flow for Jordanian and German roundabouts with comparable geometric data. It can be seen that for low circulating flow both models yield similar capacities. However, as traffic flow increases, the Jordanian model (Equation 2.18) provides higher capacity values than does the German (Equation 2.3), with the maximum difference being 100 pcu/h. The authors attribute this to differences in driving behaviors in the two countries.

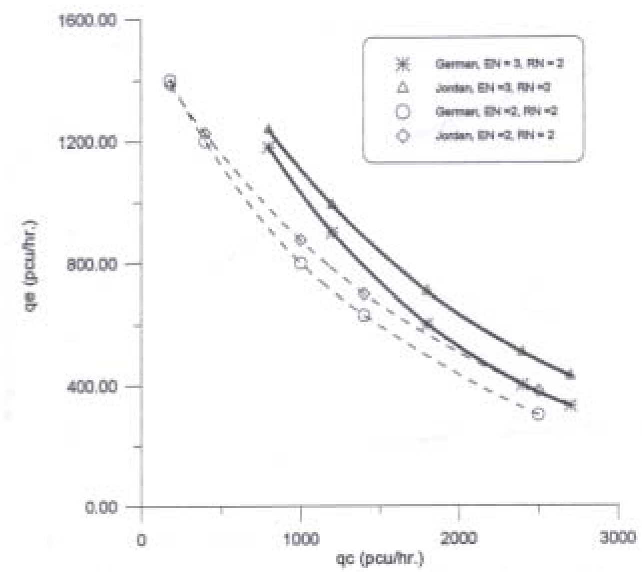


FIGURE 2.18 Jordanian and German Capacity Values for Different Numbers of Entry Lanes (EN) and Circulating Roadway Lanes (RN) (Al-Masaeid and Faddah, 1997)

d. Al-Masaeid (1999)

Figure 2.19 shows the relationship between the ratio of estimated capacity to observed capacity and the circulating traffic flow for three cases. The results suggest that the Australian capacity model with a minimum headway of 0.5 s is best applicable to Jordanian conditions.

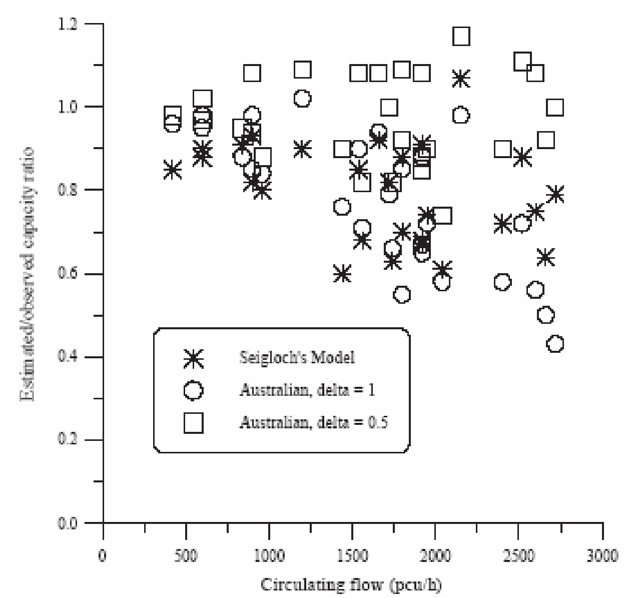


FIGURE 2.19 Performance of Australian and Siegloch Gap Theoretical Models (Al-Masaeid, 1999)

e. Hagring (2000)

Hagring studied the OD model proposed by Akçelik et al. (1996) and Akçelik (1997), which reduces the basic gap acceptance capacity by a factor that takes into account the origin-destination pattern. The model proposed by Akçelik differs from that proposed by Hagring in that it is also applicable to one-lane roundabouts and is symmetric, that is, it makes no distinction between which of the approaches is dominant. Figure 2.20 compares the capacities estimated by an ordinary gap acceptance model with those estimated by the Akçelik model. The plot, based upon Swedish roundabout measurements, indicates that the SR45/AUSTROADS method (Troutbeck 1989) overestimates capacity. The effects on capacity given by the Akçelik OD model depend not only on the OD pattern but also on the proportion of queued vehicles, and so the overestimation of capacity shown in Figure 2.20 does not depend solely on the OD flow.

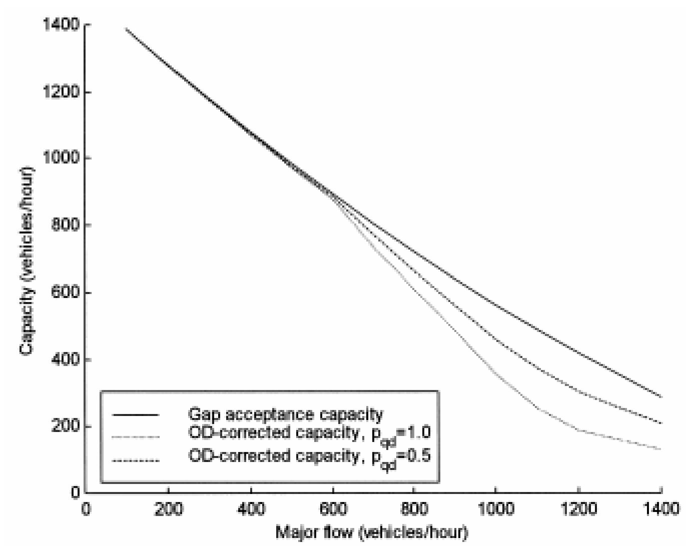


FIGURE 2.20 Predicted Capacity from Ordinary Gap Acceptance Model and Akçelik Model (Hagring, 2000)

f. Flannery, Elefteriadou, Koza, and McFadden (1998)

To evaluate the accuracy of the SIDRA software in predicting delay for U.S. single-lane roundabouts, a comparison of the field-measured control delay with that predicted by the software is shown in Figure 2.21. SIDRA appears to accurately predict delay for sites with lower volumes, whereas for sites with higher volumes, the software tends to underestimate delay. The authors concluded that SIDRA is valid for predicting delay in U.S. roundabouts and attributed the discrepancy at higher volumes to differences between Australian and U.S driving behavior.

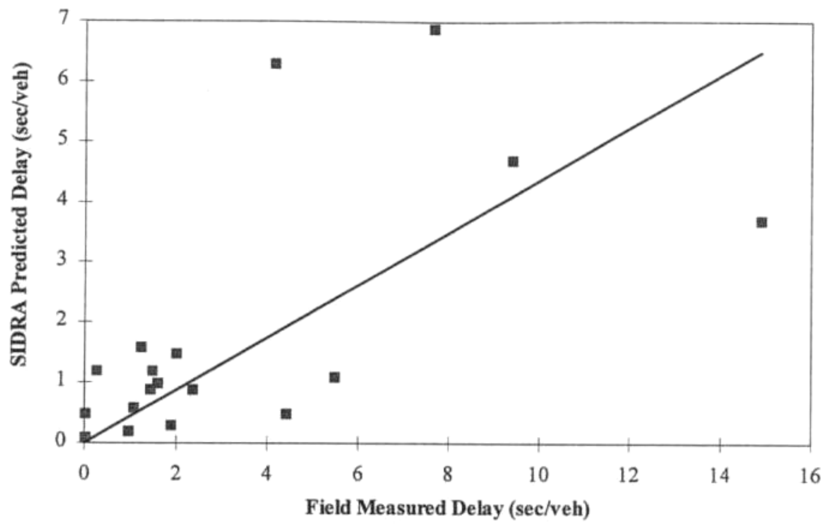


FIGURE 2.21 Comparison of Field-measured Delay and SIDRA-predicted Delay (Flannery et al., 1998)

g. Flannery, Kharoufeh, Gautam, and Elefteriadou (2000)

The Troutbeck (1993) model estimates service time as the inverse of approach capacity and its variance is indirectly accounted for by assuming deterministically or exponentially distributed service times. Comparisons between the Troutbeck and developed models are presented in Figures 2.22 and 2.23. Figure 2.22 shows the insensitive nature of the Troutbeck model as the circulating flow increases, whereas the new model predicts an increase in delay as is expected. Figure 2.23 plots the differences between field and estimated delay and shows that both models initially under-predict delay. With increasing circulating flow, however, the new model appears to predict delay more accurately than the Troutbeck method.

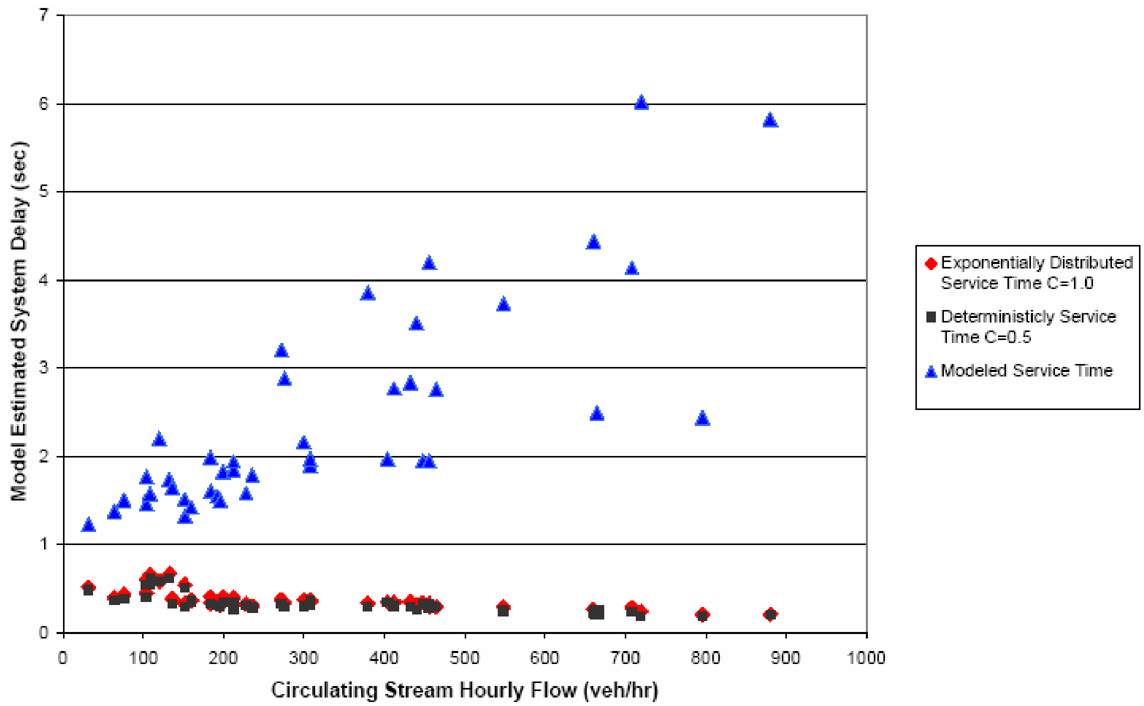


FIGURE 2.22 Model-estimated System Delay Versus Conflicting Hourly Flow (Flannery et al., 2000)

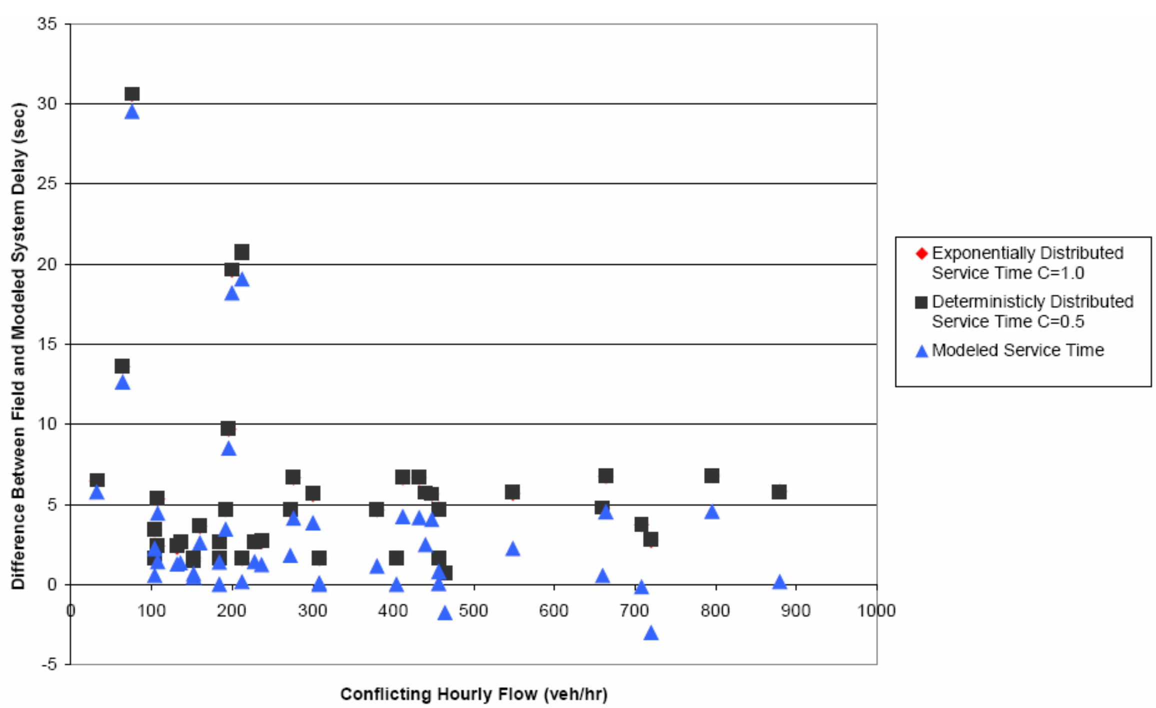


FIGURE 2.23 Field System Delay Versus Model-estimated System Delay (Flannery et al., 2000)

h. Al-Omari, Al-Masaeid, and Al-Shawabkah (2004)

The measured stopped delay was regressed against both the M/M/1 and SIDRA models. Figures 2.24 and 2.25 show plots between actual and predicted delay times using the M/M/1 and SIDRA models, respectively. Both models show high variability, particularly in the upper delay range. The follow-up headway and critical gap times are the primary parameters used by the SIDRA model to estimate delay. These parameters were calibrated and the estimated versus observed delay times were plotted as shown in Figure 2.25. The SIDRA values again show high variability, especially for greater delay. The authors conclude that the large difference between the SIDRA and observed delay values is due to the difference in driving behavior between Jordanian and Australian drivers.

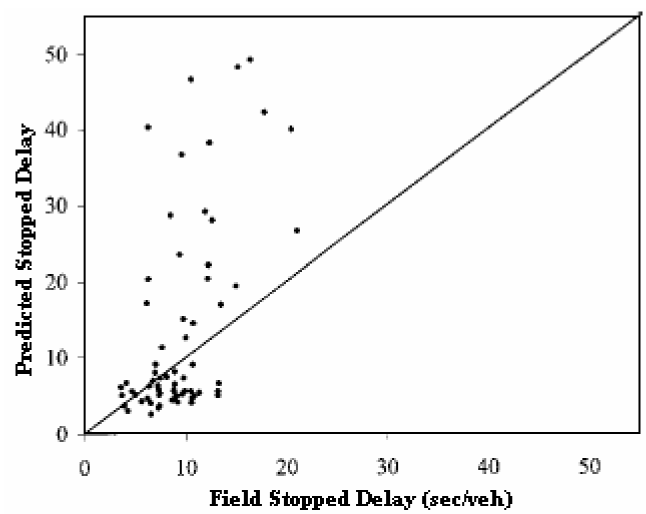


FIGURE 2.24 M/M/1 Model versus Field Stopped Delay (Al-Omari et al., 2004)

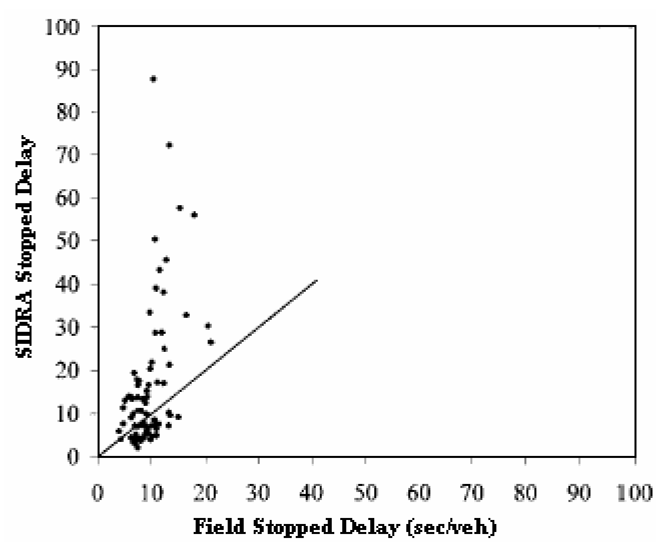


FIGURE 2.25 SIDRA versus Field Stopped Delay (Al-Omari et al., 2004)

2.3 EMISSIONS AND ENVIRONMENTAL IMPACTS OF ROUNDABOUTS

a. Várhelyi (2001)

The traffic counts showed no significant changes between volumes before and after the installations of small roundabouts.

The influence area around each junction was divided into two areas: an approach stretch and an exit stretch. Table 2.4 shows data for the junction that was formerly a signalized intersection. The delay per car decreased an average of 11 s and the number of cars stopping at the junction decreased from 63% to 26% of the total. Both CO and NO_x emissions and fuel consumption decreased on approaches and increased slightly on exits. The replacement of the signalized intersection resulted in an average decrease of 29% in CO emissions, 21% in NO_x emissions, and 28% in fuel consumption per car within the influence area.

TABLE 2.4 Average CO and NO_x Emissions and Fuel Consumption per Car and per Day at a Signalized Intersection Rebuilt as a Roundabout (Average Traffic Volume 23,500 veh/day) (Várhelyi, 2001)

Number of stretches	Per car			Per day
	Approach 4	Exit 4	Total 8	
Number of Observations				
Before	102	109	211	
After	164	167	331	
CO				
Before	2817 mg	611 mg	3428 mg	80.5 kg
After	1777 mg	650 mg	2427 mg	57.0 kg
Change	-1040 mg	39 mg	-1001 mg	-23.5 kg
	-37%*	+6%	-29%	-29%
NO_x				
Before	273 mg	109 mg	382 mg	8.9 kg
After	193 mg	107 mg	300 mg	7.0 kg
Change	-80 mg	-2 mg	-82 mg	-1.9 kg
	-29%	-2%	-21%	-21%
Fuel Consumption				
Before	17236 mg	4287 mg	21523 mg	505.7 kg
After	10913 mg	4455 mg	15368 mg	361.1 kg
Change	-6323 mg	+168 mg	-6155 mg	-144.6 kg
	-37%	+4%	-28%	-28%

* Statistically significant change ($P < 0.05$) according to the sign test.

Table 2.5 shows data for the junctions that were formerly yield-regulated. Time consumption increased for cars traveling on major streets and decreased for those traveling on minor streets. This translated into overall increased emissions and fuel consumption for major streets and decreased emissions and fuel consumption for minor streets. Considering average traffic volumes in an average junction, total CO emissions increased by 6%, NO_x emissions increased by 4%, and fuel consumption increased by 3%.

TABLE 2.5 Average CO and NO_x Emissions and Fuel Consumption per Car and per Day at an Average Junction with Yield Regulation Before and After Rebuilding (Várhelyi, 2001)

Number of stretches	Per car						Per day
	Major street			Minor street			
	Approach 15	Exit 16	Total 31	Approach 6	Exit 6	Total 12	
Number of observations							
Before	399	406	805	82	71	153	
After	548	532	1080	69	73	142	
Average traffic volume (both directions)	9700 veh/day			3130 veh/day			12830 veh/day
CO							
Before	1483 mg	1313 mg	2796 mg	1708 mg	843 mg	2551 mg	35.1 kg
After	1565 mg	1601 mg	3166 mg	1102 mg	925 mg	2027 mg	37.0 kg
Change	+82 mg	+288 mg	+370 mg	-606 mg	+82 mg	-524 mg	+1.9 kg
	+6%	+22%*	+13%*	-35%*	+10%	-20%	+6%
NO_x							
Before	238 mg	200 mg	439 mg	162 mg	146 mg	308 mg	5.2 kg
After	196 mg	278 mg	475 mg	112 mg	150 mg	262 mg	5.4 kg
Change	-42 mg	+78 mg	+36 mg	-50 mg	+4 mg	-46 mg	+0.2 kg
	-18%*	+39%*	+8%	-31%*	+3%	-15%	+4%
Fuel consumption							
Before	11563 mg	9728 mg	21291 mg	9784 mg	5601 mg	15385 mg	254.6 kg
After	11033 mg	11976 mg	23009 mg	6482 mg	5745 mg	12227 mg	261.4 kg
Change	-530 mg	+2248 mg	+1718 mg	-3302 mg	+144 mg	-3158 mg	+6.8 kg
	-5%*	+23%*	+8%	-34%*	+3%	-21%	+3%

*Statistically significant change ($P < 0.05$) according to the sign test.

Comparing the relative amount of decreased and increased emissions and fuel consumption shown in Tables 2.4 and 2.5, respectively, Várhelyi concluded that, "reductions at a rebuilt signalised [sic] intersection can "compensate for" the increase of emissions and fuel and fuel consumption at several yield regulated intersections rebuilt as roundabouts."

b. Mandavilli, Russell, and Rys (2003)

Four measures of effectiveness were obtained from the SIDRA software for use in this study: carbon monoxide, carbon dioxide, nitrogen oxides, and hydrocarbons. Statistical analysis was done separately for the AM and PM data and the results were averaged for all sites studied. Table 2.6 presents the overall results from this analysis.

TABLE 2.6 Overall Emissions Results (Mandavilli et al., 2003)

Measures Of Effectiveness	*SC**	*RA**	*% Diff.*	*Statistically Different*
		AM Results		
Carbon Monoxide (kg/hr)	9.77	7.67	-21%	Yes
Carbon Dioxide (kg/hr)	138.91	117.18	-16%	Yes
Oxides of Nitrogen (kg/hr)	0.31	0.25	-20%	Yes
Hydrocarbons (kg/hr)	0.23	0.19	-18%	Yes
		PM Results		
Carbon Monoxide (kg/hr)	11.8225	6.855	-42%	Yes
Carbon Dioxide (kg/hr)	335.7	138	-59%	Yes
Oxides of Nitrogen (kg/hr)	0.3875	0.2015	-48%	Yes
Hydrocarbons (kg/hr)	0.662375	0.23	-65%	Yes

* SC denotes AWSC or TWSC, RA denotes Roundabout

Results from the SIDRA analysis also showed a statistically significant decrease in delay, queuing, and stopping after the modern roundabouts were installed. This is evident from the decreases in all measures of effectiveness. Statistical tests also showed that all of these decreases are statistically significant. Based on these results, Mandavilli et al. concluded that the installation of modern roundabouts significantly reduced vehicular emissions by making traffic flow orderly and that a modern roundabout may be the best alternative for reducing emissions at several other Kansas intersections with ranges in traffic volume similar to those at the sites used in this study.

3.0 STUDY METHODOLOGY AND SIDRA SOFTWARE

This study uses simulation analysis to compare roundabouts with unsignalized and signalized intersections with regard to their operational performance. The unsignalized intersection used for this analysis is two-way stop controlled (TWSC) and the signalized intersection is a pre-timed signal. All intersections types evaluated have four legs and consist of a "major" road and a "minor" road. The major road volume is twice that of the minor road. The turning percentages used for this analysis are 10% left-turning, 80% through, and 10% right-turning vehicles on the major road and 20% left-turning, 60% through, 20% right-turning vehicles on the minor road. Two scenarios were developed for the analysis; the first scenario assumes a heavy vehicle (HV) percentage of 10% on both major and minor roads and the second assumes a HV percentage of 10% on the minor road and 20% on the major road.

The roundabout assumed for the analysis is a single-lane roundabout with a central island diameter of 100 ft (30.5 m) and a circulating roadway width of 18 ft (5.5 m). The terrain is assumed to be level, where the grade on all approaches is less than 2%, as this is most common in Delaware. All approaching and departing lanes are 12 ft (3.7 m) wide.

The simulation software used for this study is aaSIDRA (Signalized and unsignalized Intersection Design and Research Aid), also known as SIDRA, version 2.1. Both the U.S. Highway Capacity Manual 2000 (HCM) and the Federal Highway Administration (FHWA) Roundabout Guide recognize the SIDRA software package for the analysis of intersection capacity, level of service, and other performance characteristics. SIDRA uses an empirical gap-acceptance method to model roundabout performance that takes into account both the roundabout geometry and driver behavior and is applicable for both left-hand and right-hand driving. The software is capable of analyzing signalized intersections (actuated and fixed-time), unsignalized intersections including two-way stop-controlled, all-way stop-controlled, and yield-controlled intersections, and roundabouts.

The SIDRA default roundabout critical gap and follow-up time values are not fixed and are estimated as functions of the roundabout geometry, circulating flow, and entry lane flows. The software package offers the option to use the HCM capacity model as well as default gap-acceptance values based upon HCM parameters for two-way stop sign controlled intersections and the HCM delay and queue equations for signalized and two-way stop-sign controlled intersections. Analysis based on SIDRA is therefore aptly suited to U.S. conditions. The SIDRA capacity model may also be calibrated as to better reflect road and local driver characteristics. Consequently, the roundabout analysis conducted in this study has been calibrated for local driver characteristics using critical gap and follow-up times determined from the data collection.

4.0 DATA COLLECTION AND REDUCTION

Two Maryland single-lane roundabouts near the Delaware border were studied in early 2005 in order to collect data. One site is located at the intersection of MD 273 and MD 276 in Cecil County and the other at the intersection of MD 18 and Castle Marina Road in Queen Anne's County. At each site, the approach with the highest volume entering the roundabout and the highest conflicting volume around the circle was identified and videotaped. The two locations were chosen for their similarities to each other in geometric and traffic characteristics, as well as their proximity to Delaware. Table 4.1 presents traffic and geometric information provided by the Maryland State Highway Administration (SHA) prior to data collection.

TABLE 4.1 Traffic and Geometric Data

Site	*AADT (vpd)*	*Outer/Inner Diameter (ft)*	*Date of Construction*
Cecil County, MD	16,350[1]	140 / 86	Fall 2002
Queen Anne's County, MD	14,000[2]	130 / 30	December 1999

[1]*collected in 2000*
[2]*collected in 1999*
Source: Maryland SHA

The videotapes were reviewed in order to extract data consisting of rejected lags and gaps, accepted lags and gaps, and follow-up times. The lag was determined as the difference between the time at which an entering vehicle arrived at the front of the queue and the time at which the next circulating vehicle passed this location. The gap is defined as the difference in time between the passages of two consecutive circulating vehicles at a point in front of the entering queue. The follow-up time was determined as the time difference between the passage of an entering vehicle into the roundabout and the arrival of the subsequent entering vehicle to the front of the queue, recorded when a queue of at least three entering vehicles existed. The effects of circulating vehicles exiting the roundabout at the approach of interest are not within the scope of this study and so these vehicles were ignored in the data reduction.

Figures 4.1 and 4.2 show the critical gap and follow-up times determined for the two study sites, respectively. The critical gaps were determined following Drew's method for accepted and rejected gap times. The critical gaps found are slightly below the lower limit of 4.1 s recommended by the HCM 2000. Table 4.2 presents a summary of the critical gap and follow-up times found for each site. The operating speed of circulating vehicles was determined from the videotapes to be approximately 26 mph and therefore according to the model of Polus and Shmueli, the relevant gap is 8.2 s. That is, gaps larger than 8.2 s are accepted by all entering vehicles and are not relevant in determining the critical gap.

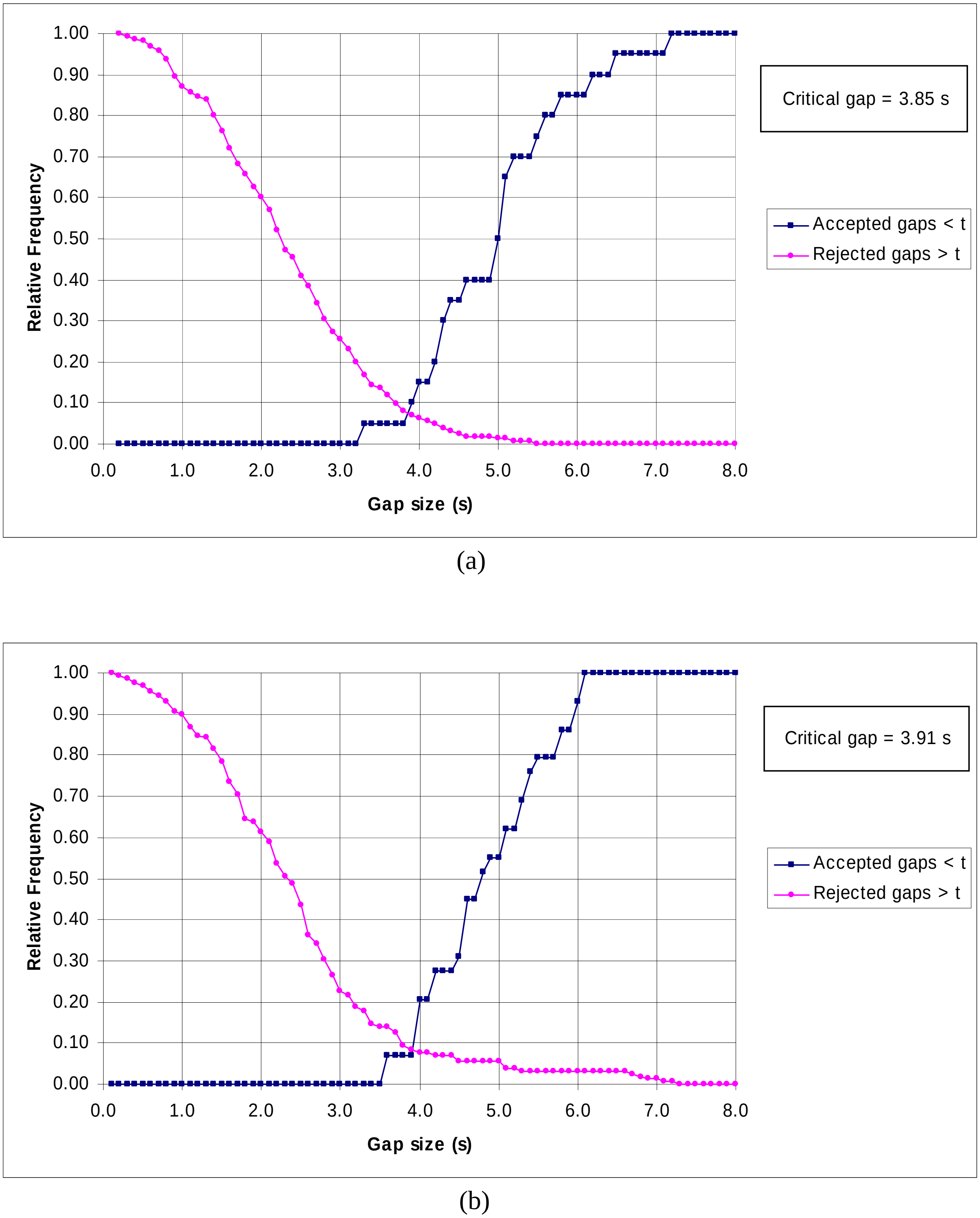
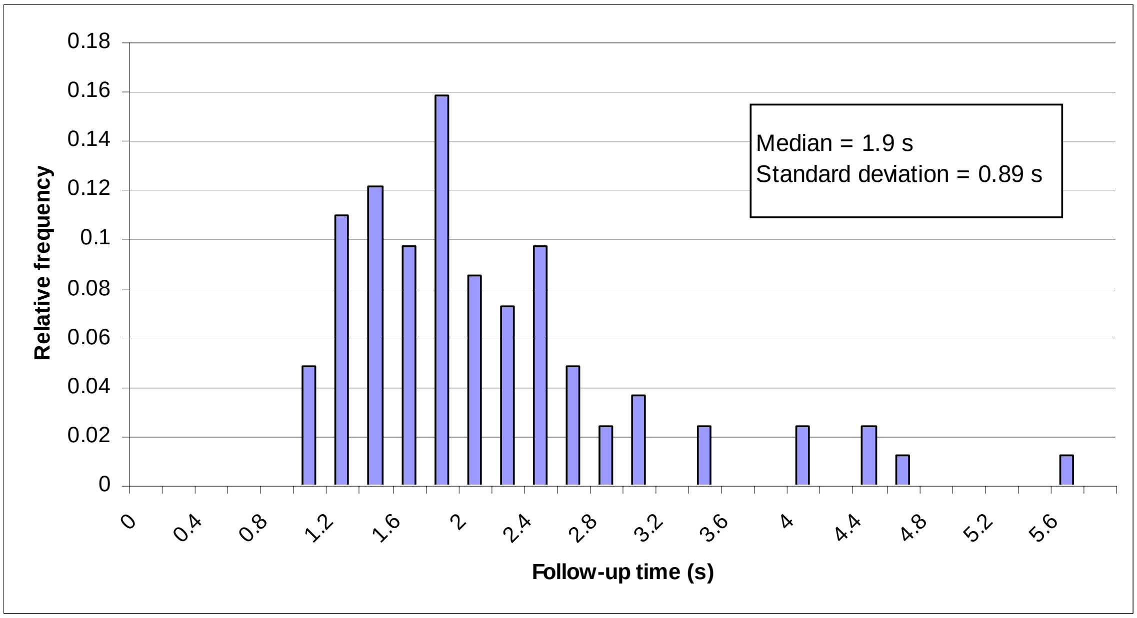
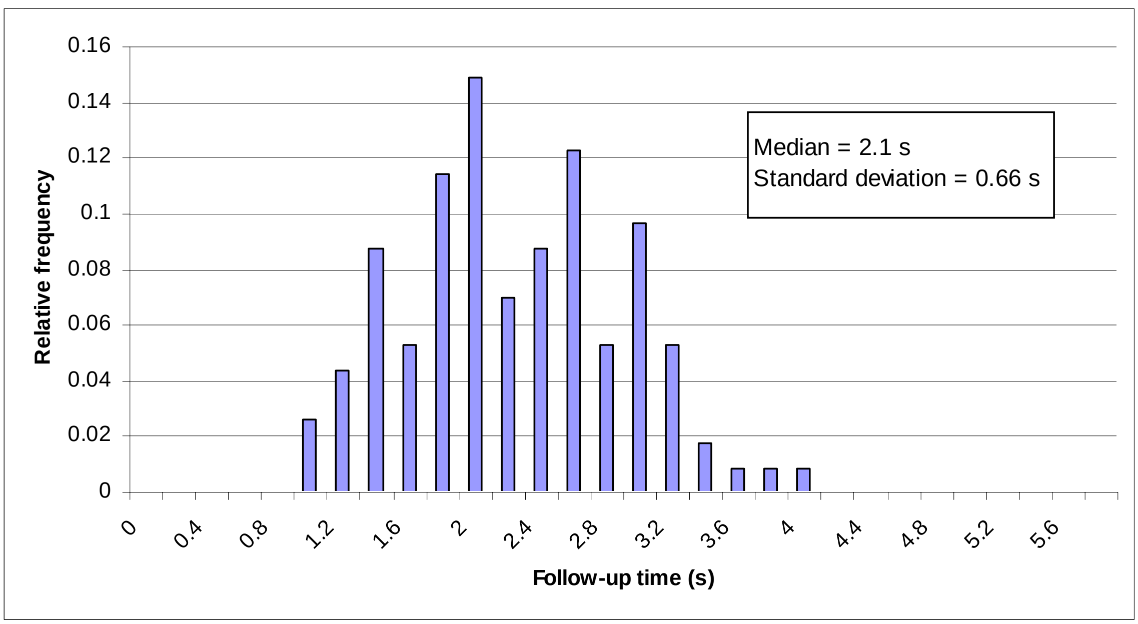


FIGURE 4.1 Determination of Critical Gap for (a) MD 273/MD 276 (Cecil County) and (b) MD 18/Castle Marina Road (Queen Anne's County)



(a)



(b)

FIGURE 4.2 Distributions of Follow-up Time for (a) MD 273/MD 276 (Cecil County) and (b) MD 18/Castle Marina Road (Queen Anne's County)

TABLE 4.2 Critical Gaps and Follow-up Times for Study Sites

Site	*Critical Gap (s)*	*Follow-up Time (s)*
MD 273/MD 276	3.85	1.9
MD 18/Castle Marina Road	3.91	2.1

5.0 PRELIMINARY ANALYSIS USING SIDRA DEFAULT GAP ACCEPTANCE VALUES

A preliminary analysis using the SIDRA software default gap acceptance parameters was conducted prior to data collection. The analysis compared the performances of a two-way stop controlled (TWSC) intersection, a roundabout, and a pre-timed un-actuated signal. Comparisons were based on the following measures of effectiveness (MOE): effective intersection capacity, major road capacity, minor road capacity, major road delay, minor road delay, major road 95% queue length, minor road 95% queue, and emissions (CO, CO_2, HC, and NO_X).

All intersections consist of four single-lane approaches. The analyses assume 12-ft wide lanes and no grade on all approaches. The roundabout inner diameter and circulating roadway width were assumed to be 100 ft and 18 ft, respectively.

Two roads, a “major road” and a “minor road,” were designated, where the total entering volume of the minor road was 50% of that of the major road in all cases. Assumed turning percentages were as follows: 10% right-turning, 80% through, and 10% left-turning vehicles on the major road and 20% right-turning, 60% through, and 20% left-turning vehicles on the minor road. For the TWSC, major road vehicles are given the right of way while the minor road vehicles are controlled by a stop sign.

Two sets of tests were conducted. The first assumed 10% heavy vehicles (HV) on both roads. The second increased heavy vehicles on the major road to 20% while heavy vehicles on the minor road remained at 10%.

5.1 CAPACITY

The initial tests with heavy vehicle percentages of 10% on both roads studied four volume scenarios: major road one-way volumes (MRV) of 100, 500, 1000, and 1250 vph. The initial tests with heavy vehicle percentages of 20% on the major road tested MRVs of 100, 300, 500, and 700 vph. Effective intersection capacity results are shown in Figure 5.1. It can be seen that in both cases the capacity of the TWSC intersection continually decreases with increasing demand volume. Also in both cases, the roundabout initially provides greater capacity than the signal but is surpassed by the signal at greater volumes. This switch occurs between MRV 1000 and 1250 vph for 10% HV on both roads and between MRV 500 and 700 vph for 20% HV on the major road. As these MRVs were arbitrarily chosen for preliminary testing, further tests were carried out later on for many more volumes in order to obtain more accurate points of switch in advantage between intersection types.

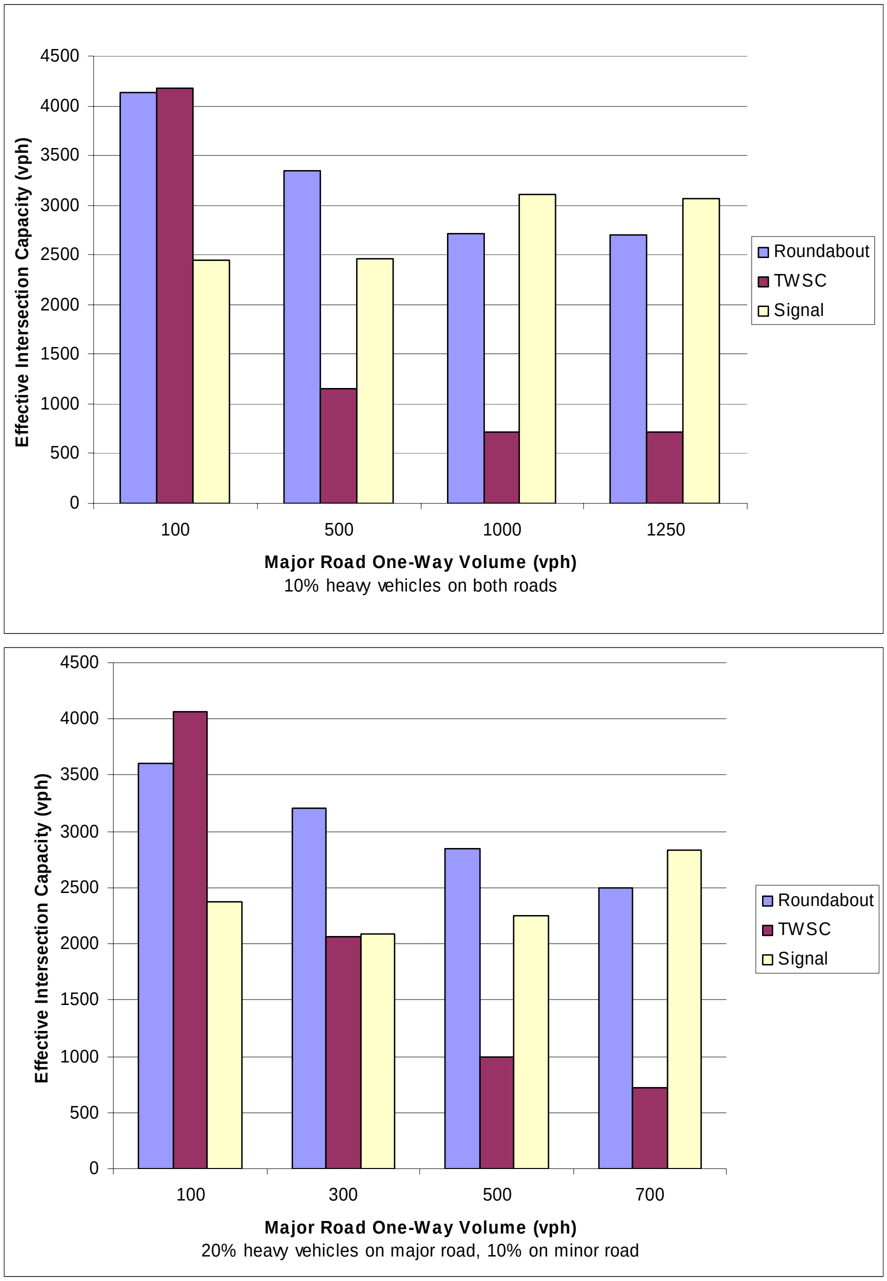
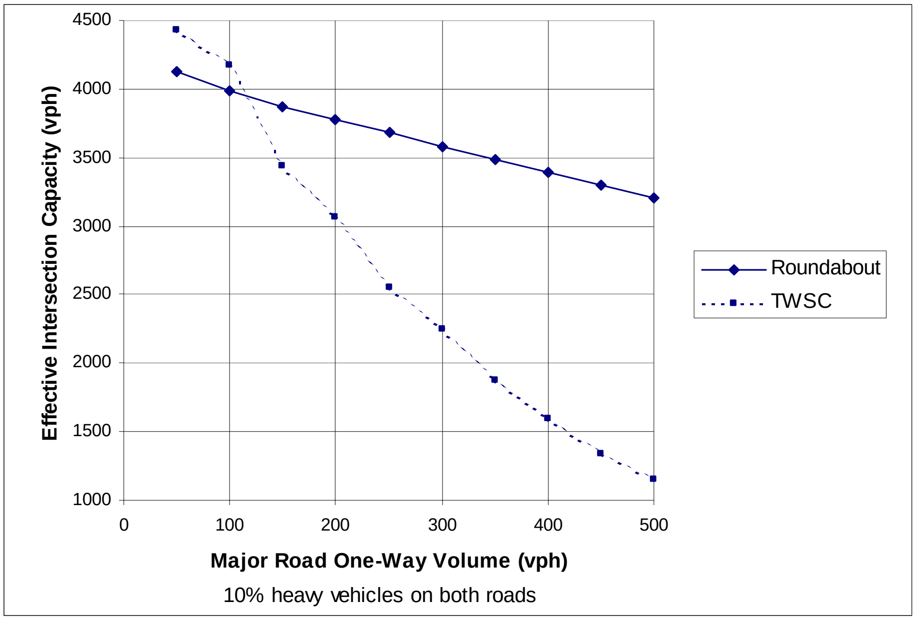


FIGURE 5.1 Initial Effective Intersection Capacity Tests for Both Heavy Vehicle Scenarios

From these observations, the following may be stated: between the three intersection types, the TWSC intersection provides the best intersection capacity for lighter volumes, a signalized intersection provides the best intersection capacity for heavier volumes, and roundabouts provide the best intersection capacity for a mid-range of volumes. To further explore this, additional tests were conducted in order to obtain better estimates of these threshold volumes. The analyses were re-focused on comparing roundabouts versus TWSC intersections and roundabouts versus signalized intersections. Figure 5.2

shows the crossover volume between a roundabout and a TWSC intersection for both heavy vehicle scenarios. Figure 5.3 shows the same between a roundabout and a signalized intersection.



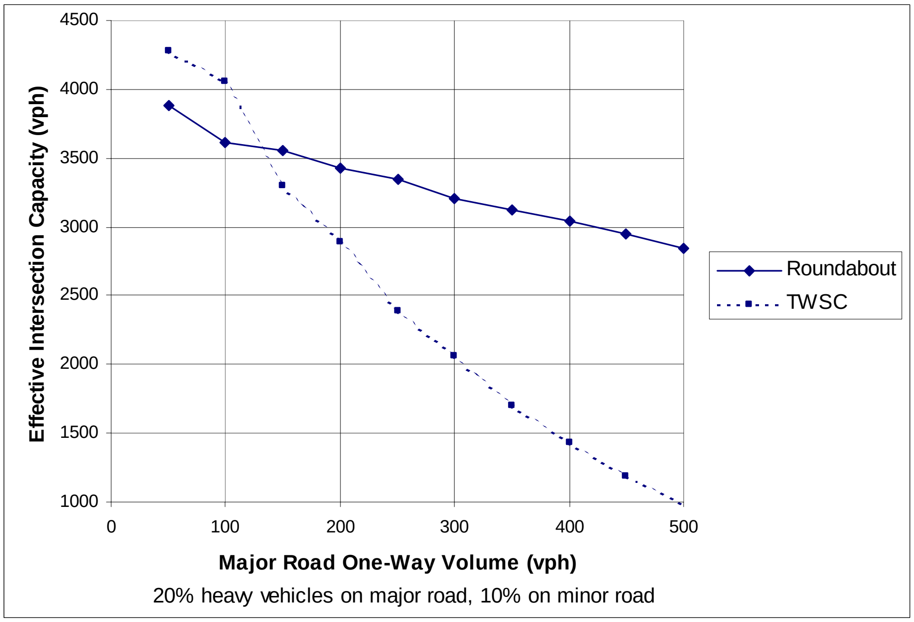


FIGURE 5.2 Intersection Capacity Threshold Volumes, Roundabout vs. TWSC Intersection

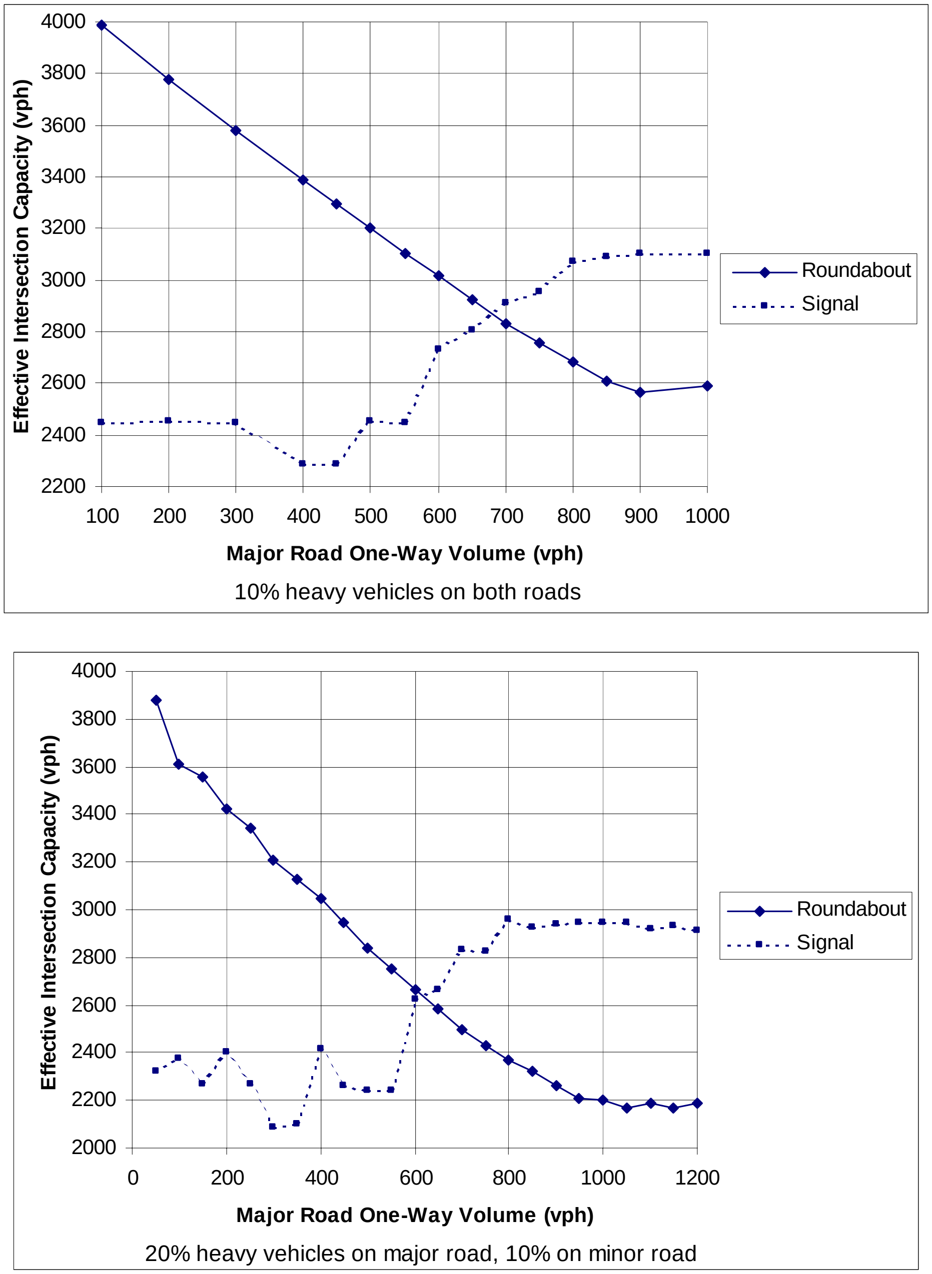


FIGURE 5.3 Effective Intersection Capacity, Roundabout vs. Signalized Intersection

Similar comparisons were made for the major and minor approach capacities. Figure 5.4 shows that with an increase in heavy vehicles, the major road capacity threshold volume between roundabouts and signalized intersections is decreased. A similar comparison was not conducted between a roundabout and a TWSC intersection, as it is irrelevant because of the theoretical zero delay experienced by major road vehicles at a TWSC intersection.

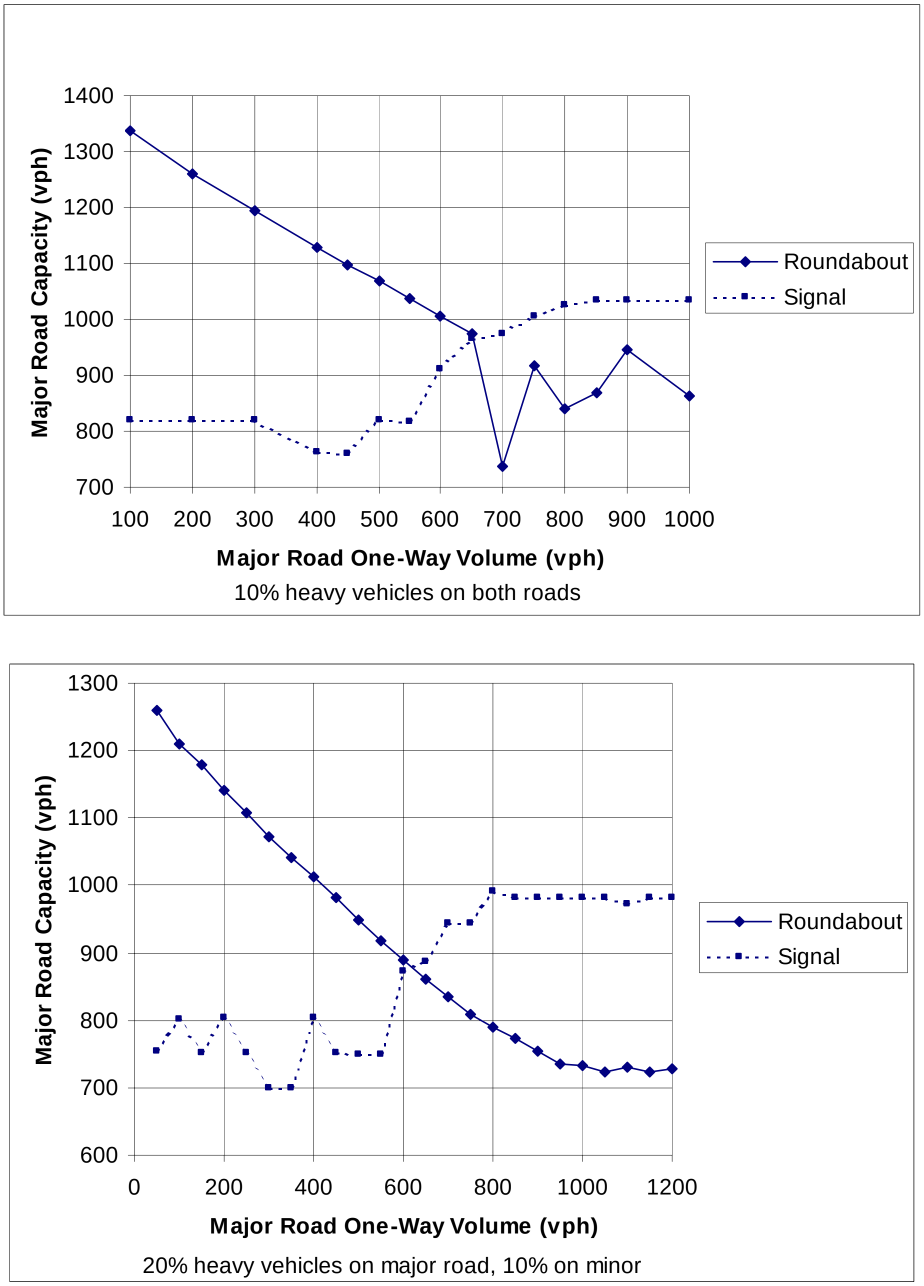
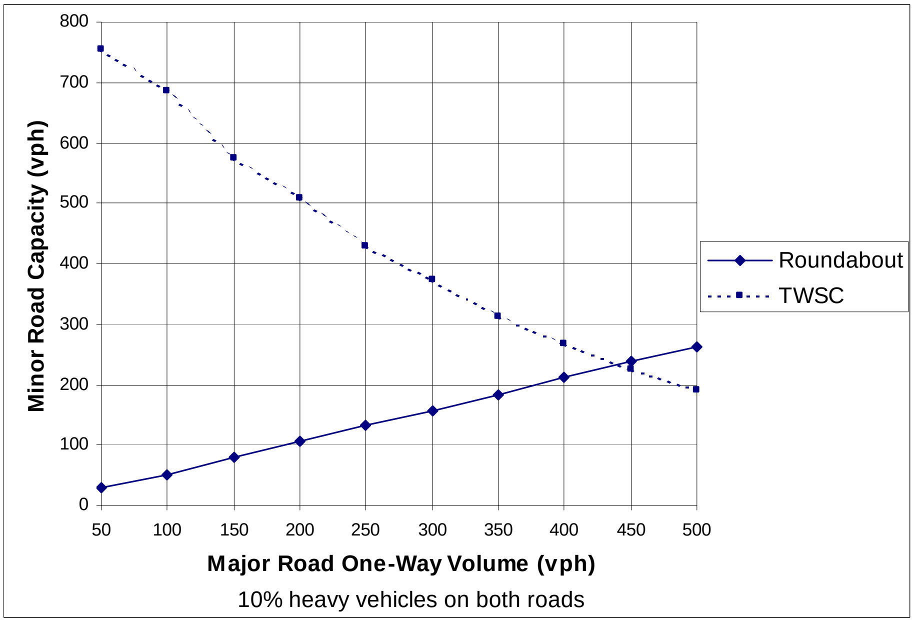


FIGURE 5.4 Major Road Capacity, Roundabout vs. Signalized Intersection

Figure 5.5 shows that for HV 10% on both roads, the minor road capacity advantage switches from the TWSC intersection to a roundabout at MRV 440 vph, whereas when the HV is increased to 20% on the major road, the roundabout always provides better minor road capacity. Figure 5.6 shows that with increased heavy vehicles, the minor road capacity threshold between roundabouts and signalized intersections increased from MRV 653 to 728 vph.



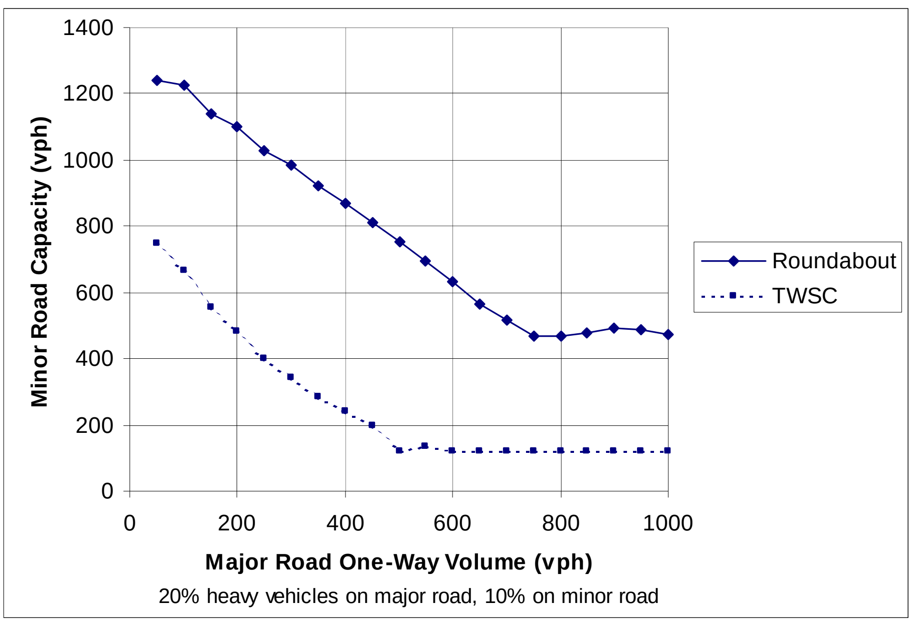
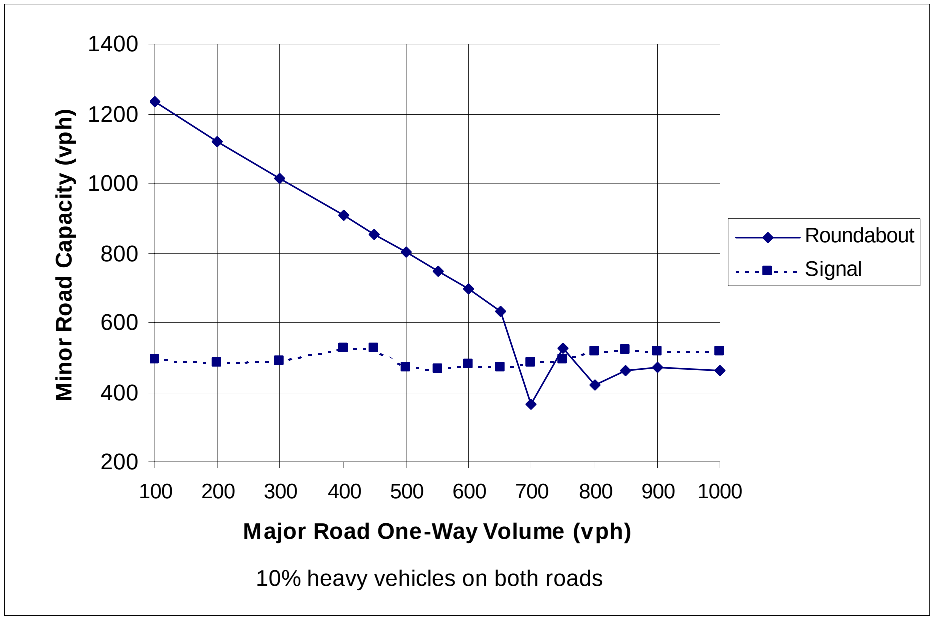


FIGURE 5.5 Minor Road Capacity, Roundabout vs. TWSC Intersection



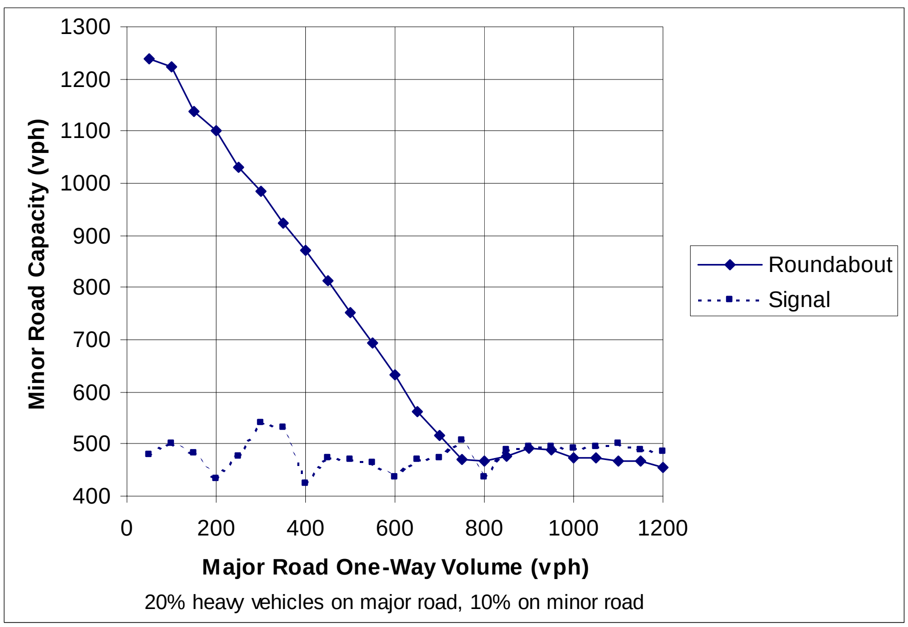
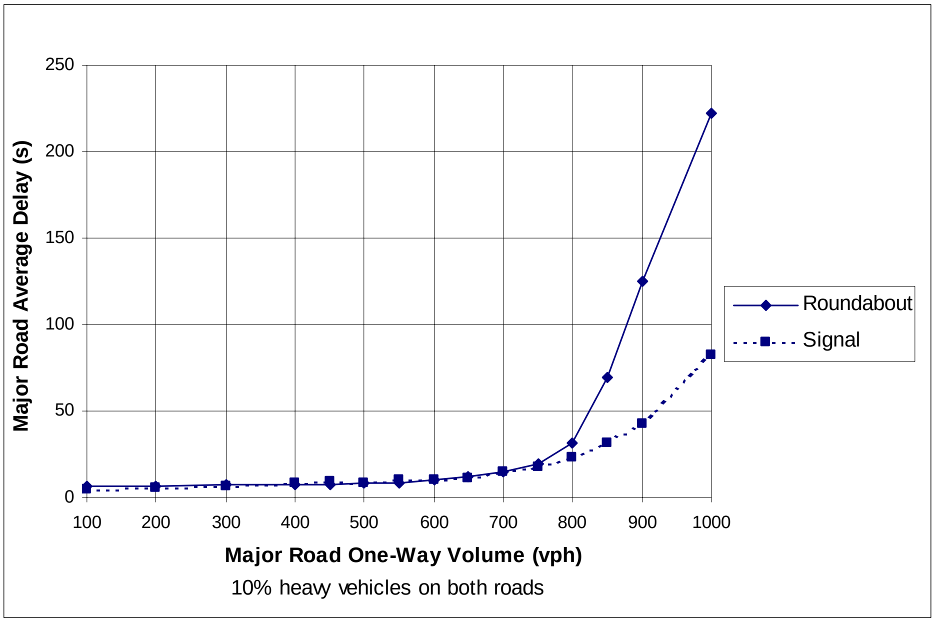


FIGURE 5.6 Minor Road Capacity, Roundabout vs. Signalized Intersection

5.2 DELAY

The SIDRA software results provide average delay times for each intersection approach. SIDRA delay estimates include all delays experienced by vehicles arriving during the demand flow period, even if some of these vehicles depart after the analysis period. The standard SIDRA default method uses the control delay, which consists of overall and geometric delay. The control delay includes deceleration and acceleration delays for the major stop experienced by queued vehicles, as well as the geometric delays experienced by all vehicles in negotiating the intersection. The HCM Delay and Queue option offered by the software will cause delay and queue lengths to be calculated using the HCM equations where applicable. The SIDRA analyses conducted for this study utilized the HCM Delay and Queue option.

Figures 5.7 and 5.8 show the points of change in advantage between roundabouts and signalized intersections for major road delay. Figure 5.7 shows both an overall and a smaller-scaled graph for conditions with 10% heavy vehicles on both roads while Figure 5.8 shows the same when heavy vehicles have been increased to 20% on the major road. It can be seen that with an increase in heavy vehicles, the major road capacity threshold is decreased from MRV 628 to 583 vph. Again, because of its irrelevance, a comparison for the major road delay between a roundabout and a TWSC intersection was not included.



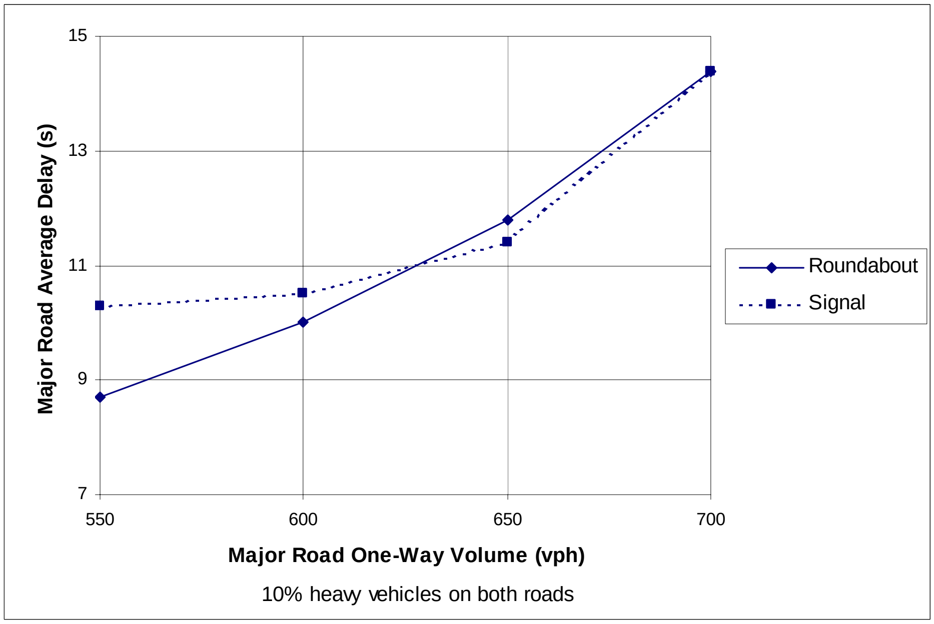


FIGURE 5.7 Major Road Delay, Roundabout vs. Signalized Intersection

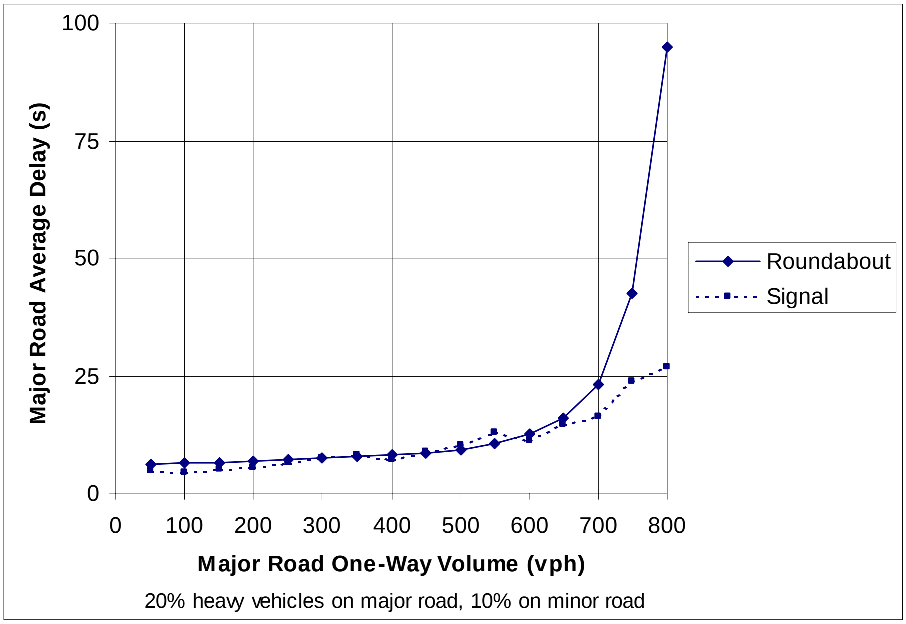


FIGURE 5.8 Major Road Delay, Roundabout vs. Signalized Intersection

Minor road delay comparisons are shown in Figures 5.9 through 5.11. As seen in Figure 5.9, in both heavy vehicle percentage cases comparing the roundabout and TWSC intersection, the roundabout experienced less delay, particularly at higher MRVs. Figure 5.10 shows the comparison between roundabout and signalized intersection for 10% heavy vehicles on both roads, and it can be seen that the threshold volume between the roundabout and signalized intersection is 830 vph. For 20% heavy vehicles on the major road and 10% on the minor road, the roundabout experiences less minor road delay for all MRVs tested.

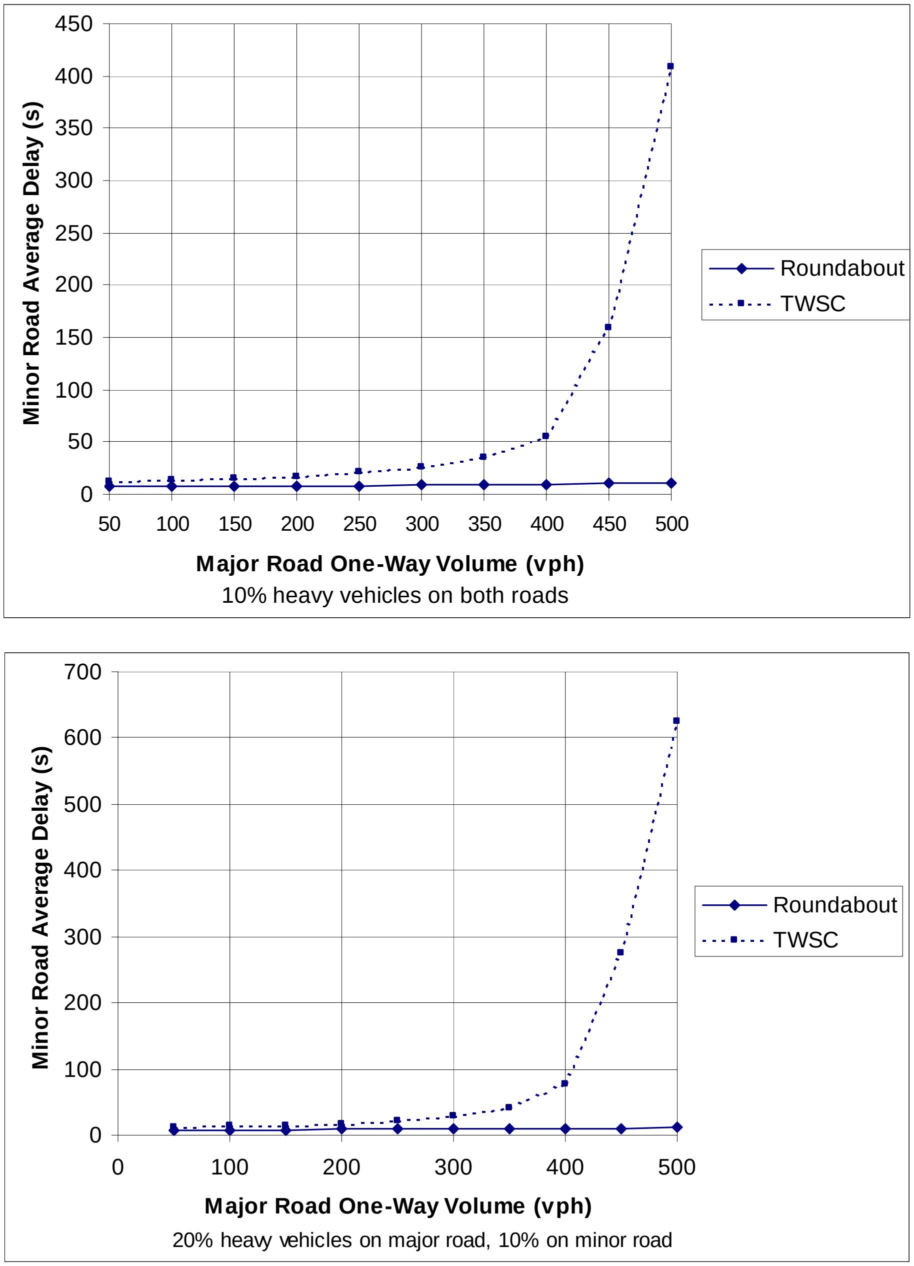
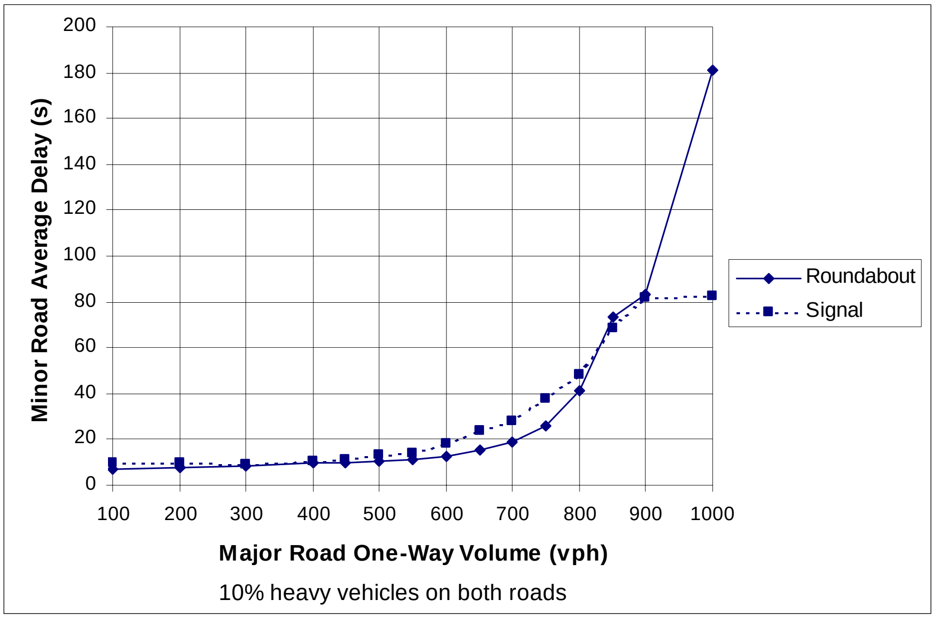


FIGURE 5.9 Minor Road Delay, Roundabout vs. TWSC Intersection



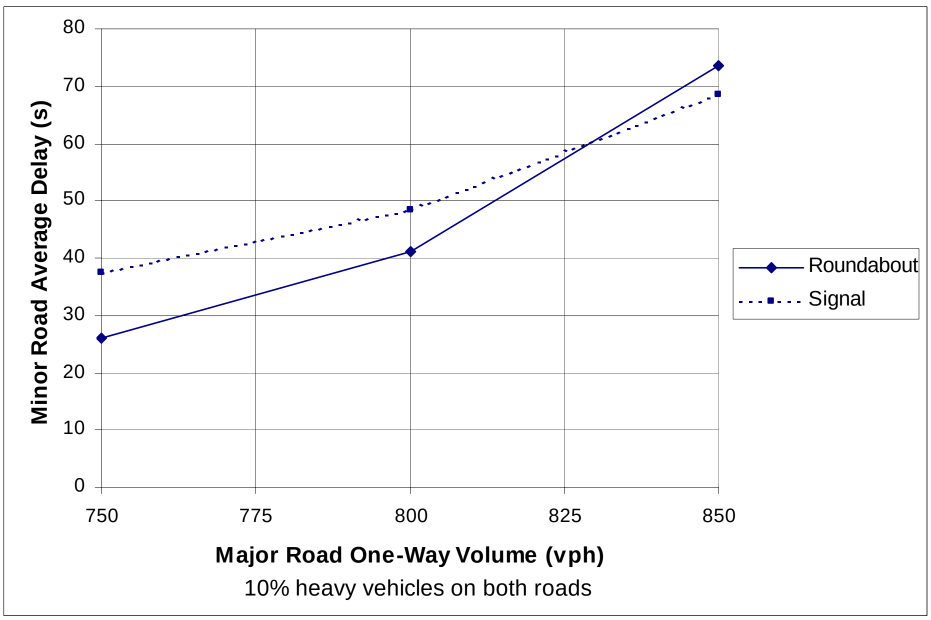


FIGURE 5.10 Minor Road Delay, Roundabout vs. Signalized Intersection

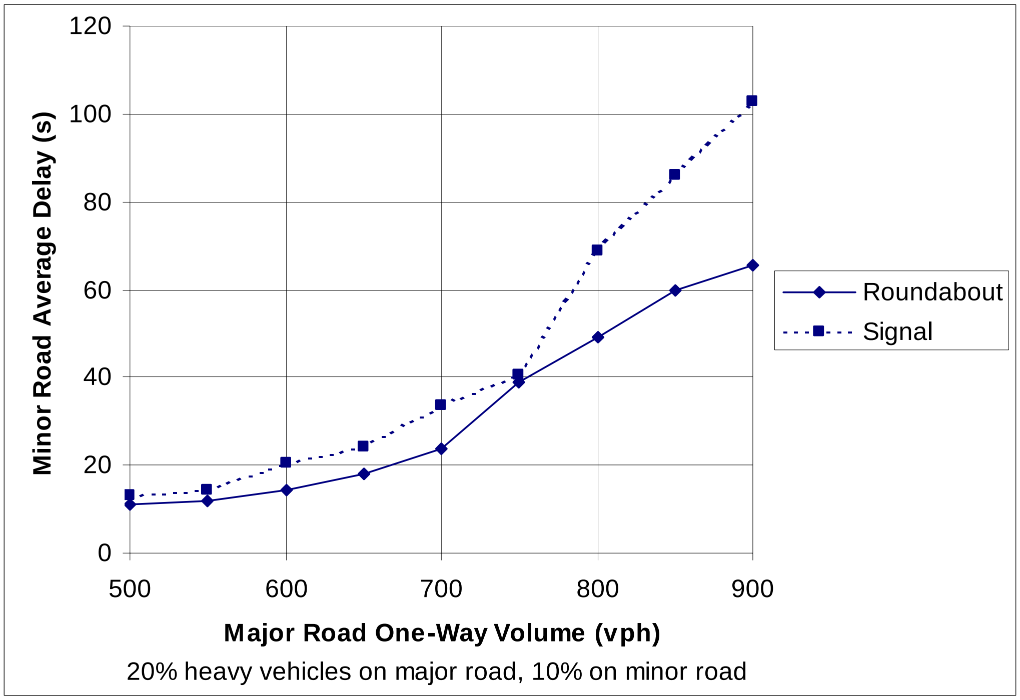


FIGURE 5.11 Minor Road Delay, Roundabout vs. Signalized Intersection

5.3 QUEUE LENGTH

The SIDRA test results provided the 95% queue lengths for each intersection approach. Figures 5.12 and 5.13 present the comparisons for major road 95% queue length for a roundabout versus a signalized intersection. For clarity, large and smaller-scaled plots are provided. It can be seen that with increased heavy vehicles, the threshold between the roundabout and signalized intersection decreases from 844 vph to 694 vph. Again, because of the irrelevance, a similar comparison for a roundabout versus a TWSC intersection was not conducted.

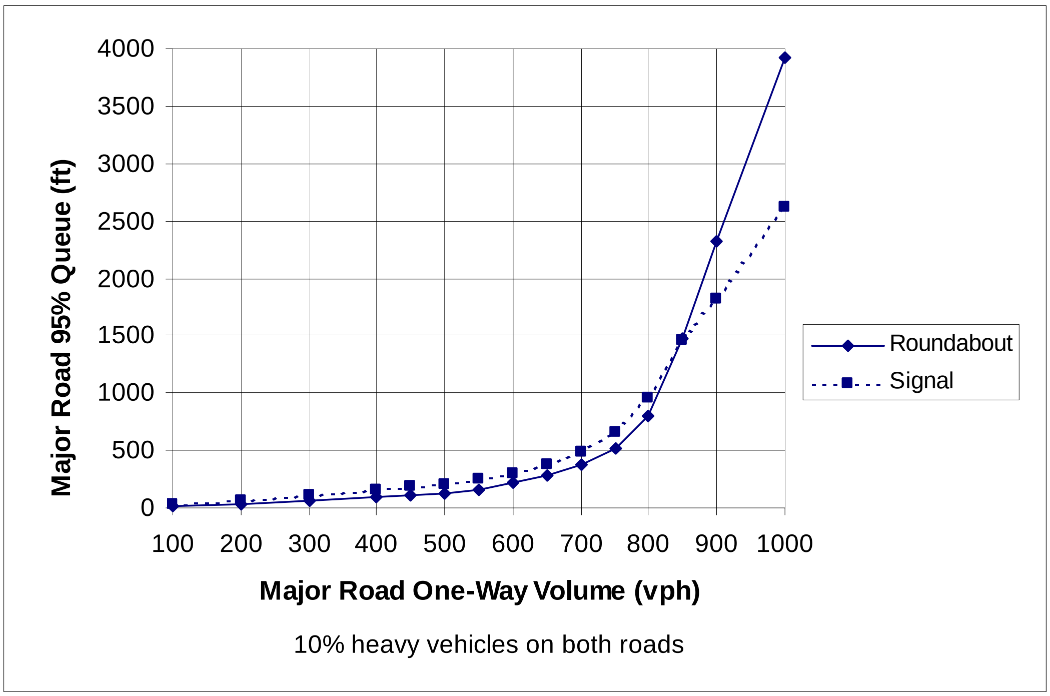
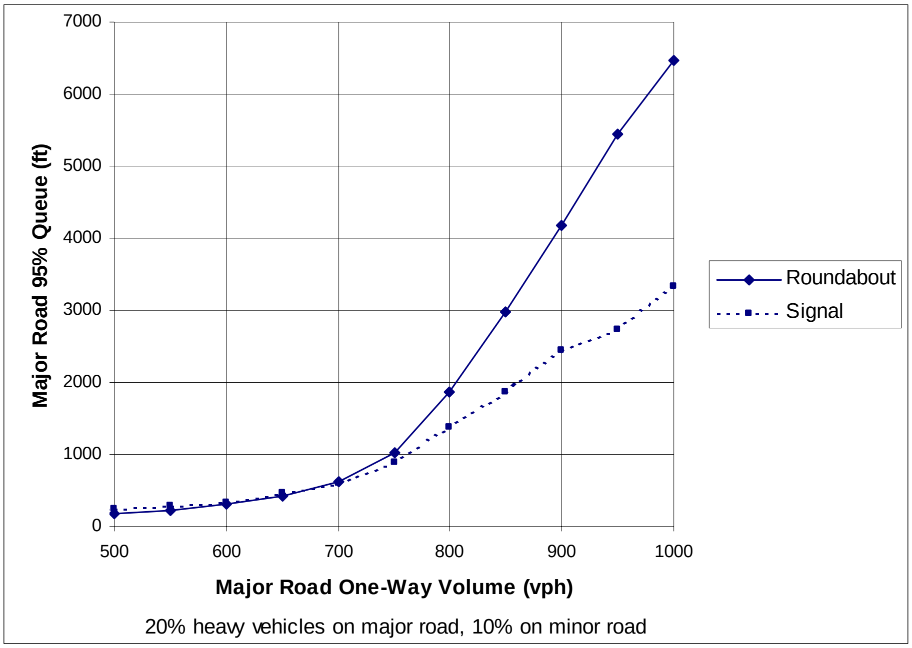


FIGURE 5.12 Major Road Queue, Roundabout vs. Signalized Intersection



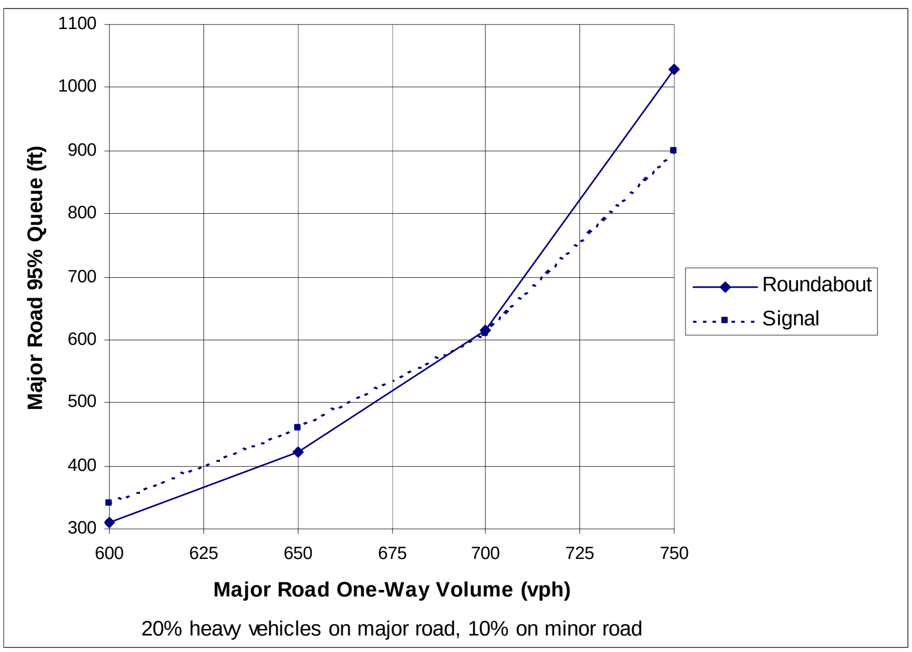


FIGURE 5.13 Major Road Queue, Roundabout vs. Signalized Intersection

The minor road 95% queue length for a roundabout versus a TWSC intersection is shown in Figure 5.14. It can be seen that in both heavy vehicle situations the roundabout provides a shorter minor road queue, though the difference in queue lengths becomes more pronounced with increased MRV and/or increased heavy vehicles.

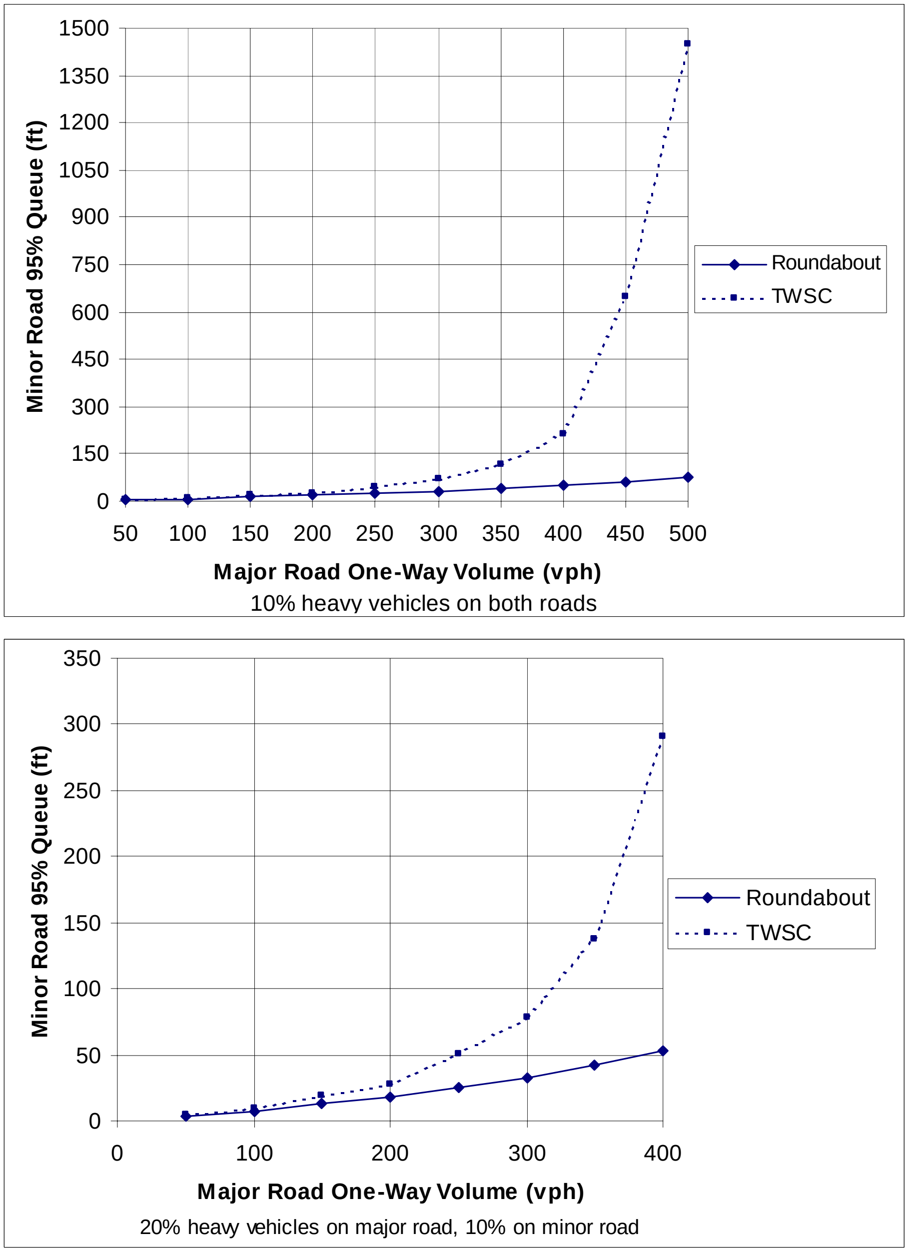
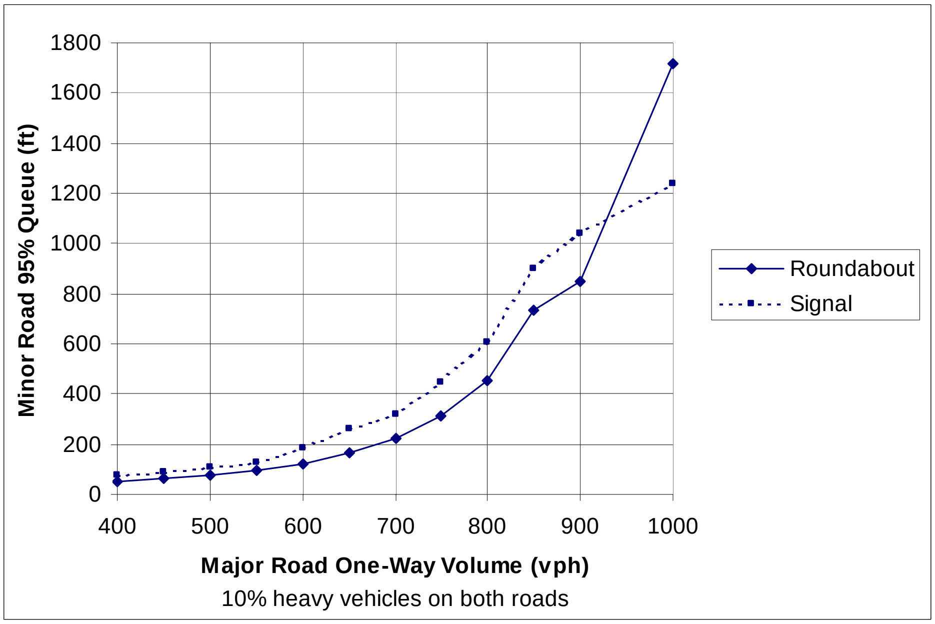


FIGURE 5.14 Minor Road Queue, Roundabout vs. TWSC Intersection

Figure 5.15 shows the minor road 95% queue comparison between a roundabout and a signalized intersection. In both cases, on the average, a roundabout experiences a shorter minor road queue up until 929 vph (10% HV both roads) and 987 vph (20% HV on major road), after which its queue length appears to increase relatively rapidly.



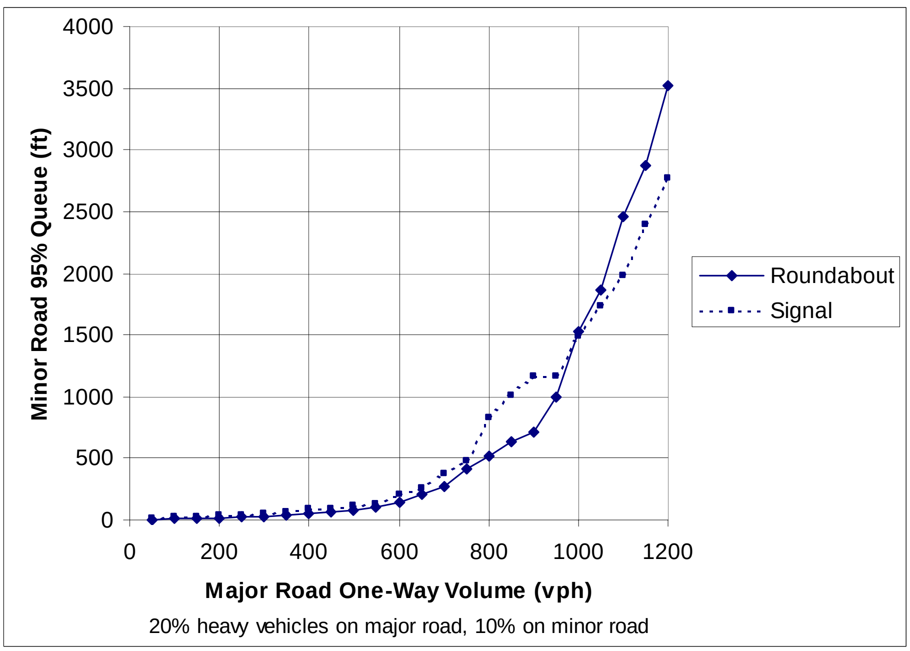


FIGURE 5.15 Minor Road Queue, Roundabout vs. Signalized Intersection

5.4 EMISSIONS

Four emissions types are included in SIDRA analysis results: HC, CO, NO_X, and CO_2. Comparisons such as those discussed above were also made between a roundabout and a TWSC intersection and between a roundabout and a signalized intersection for these four emissions.

HC Emissions

Figures 5.16 and 5.17 show HC emissions levels between roundabout and TWSC. For clarity, both overall and small-scale graphs for each analysis have been included. It can be seen that as the percentage of heavy vehicles increases, the threshold for HC emissions also increases. Above MRV 250 and 270 vph, for major road heavy vehicles of 10% and 20%, respectively, the amount of HC emissions from the TWSC increases far greater than that from the roundabout.

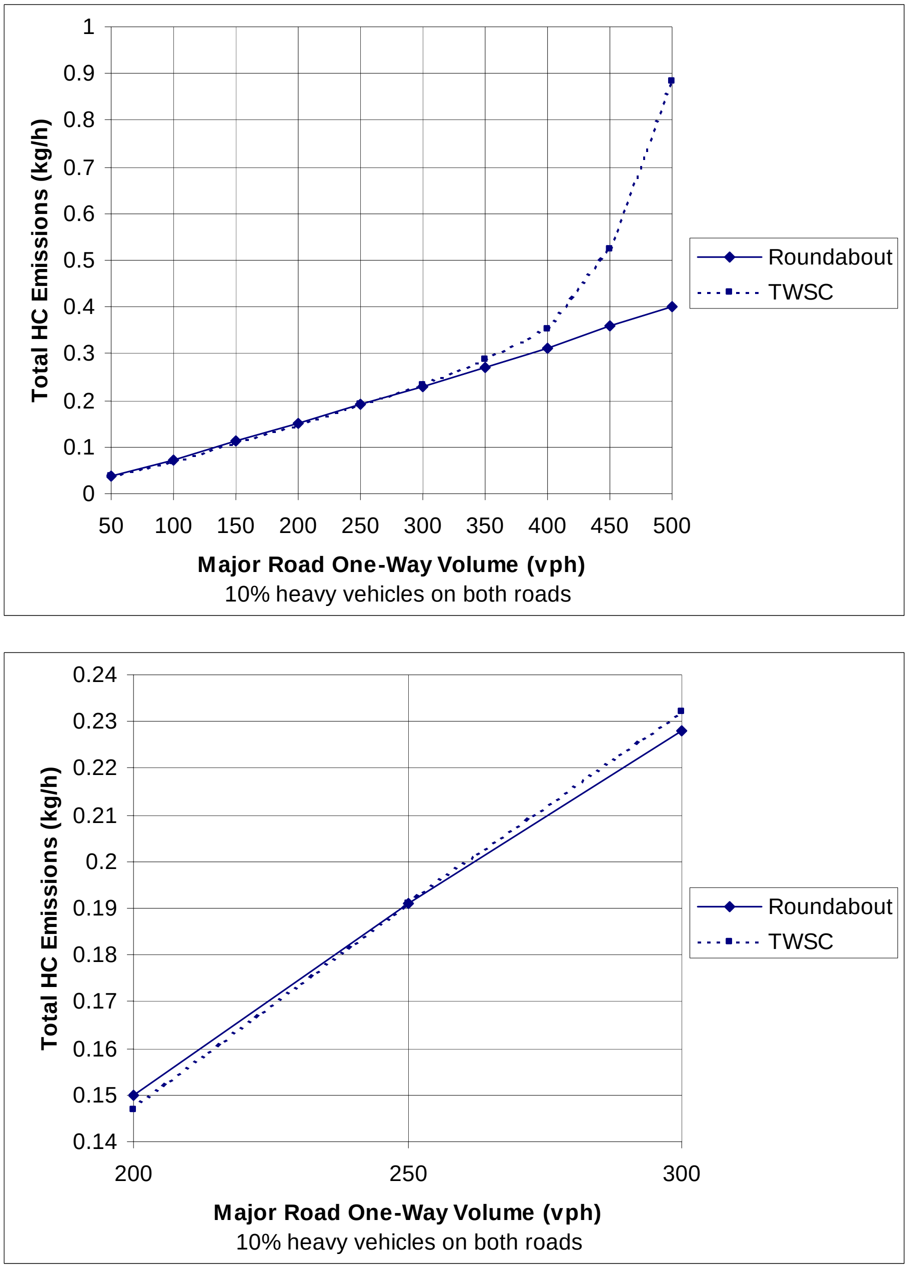


FIGURE 5.16 Total HC Emissions, Roundabout vs. TWSC Intersection

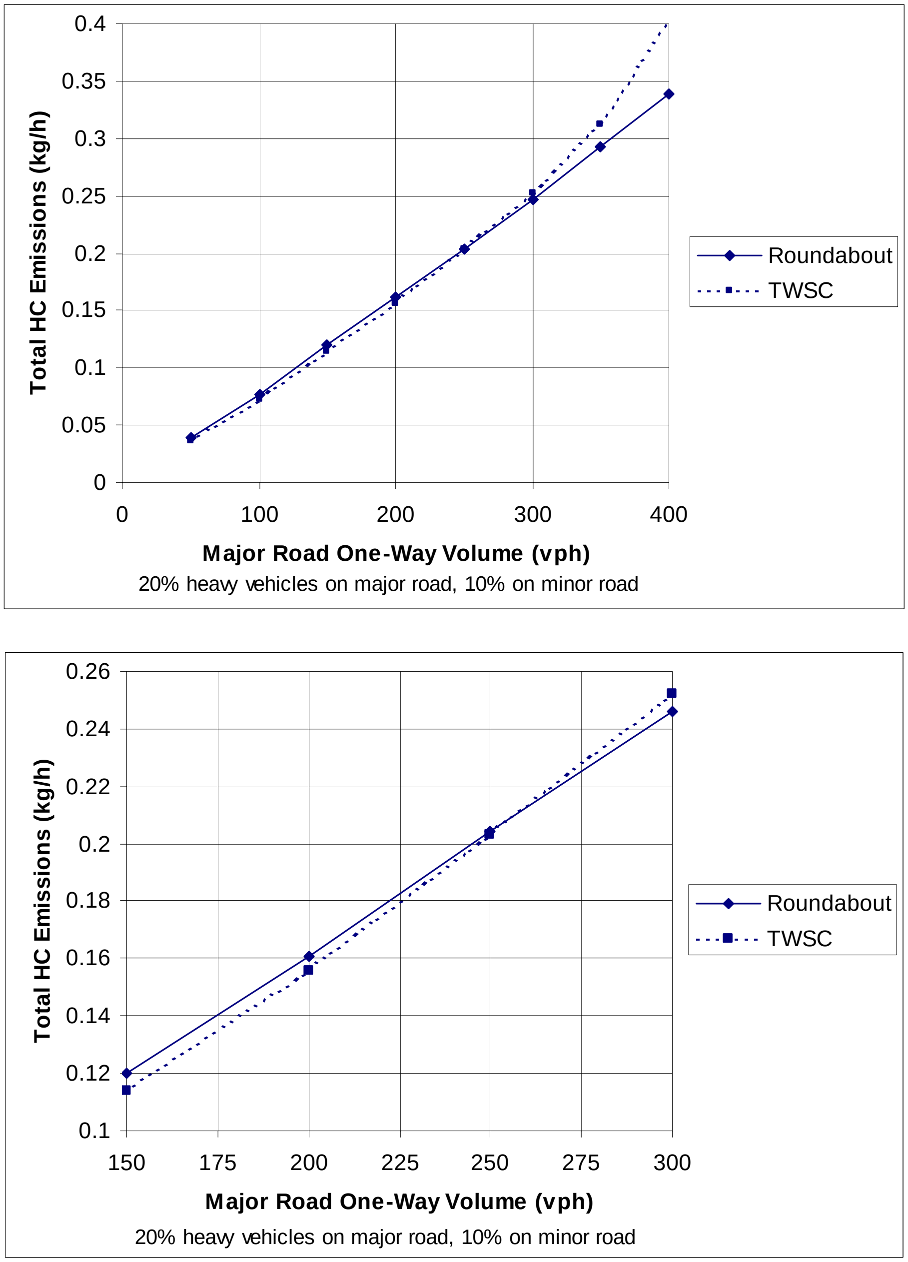
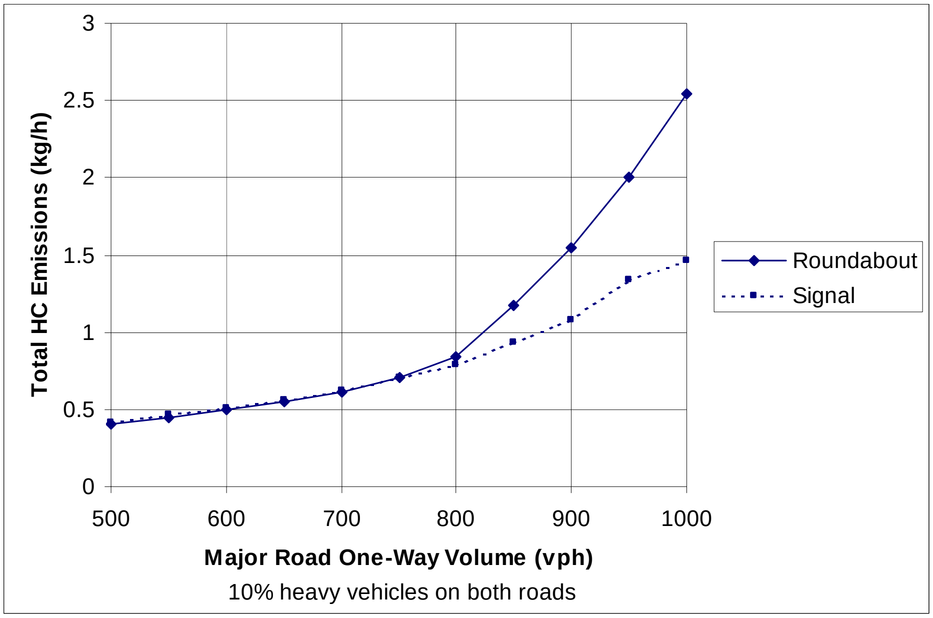


FIGURE 5.17 Total HC Emissions, Roundabout vs. TWSC Intersection

Likewise, Figures 5.18 and 5.19 shows similar graphs for a roundabout versus a signalized intersection. Here it can be seen that as the percentage of heavy vehicles increases, the threshold between a roundabout and a signalized intersection decreases from MRV 750 to 630 vph.



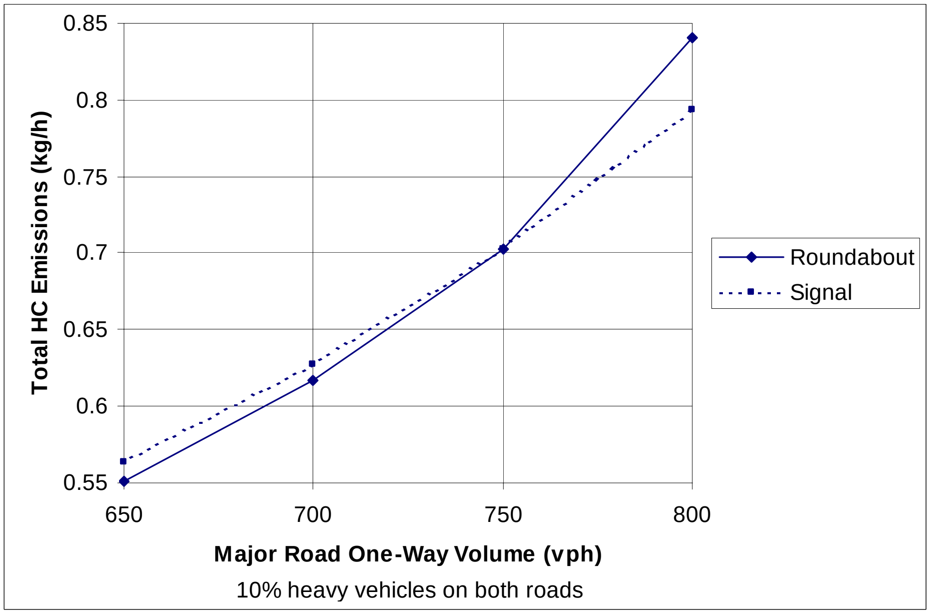


FIGURE 5.18 Total HC Emissions, Roundabout vs. Signalized Intersection

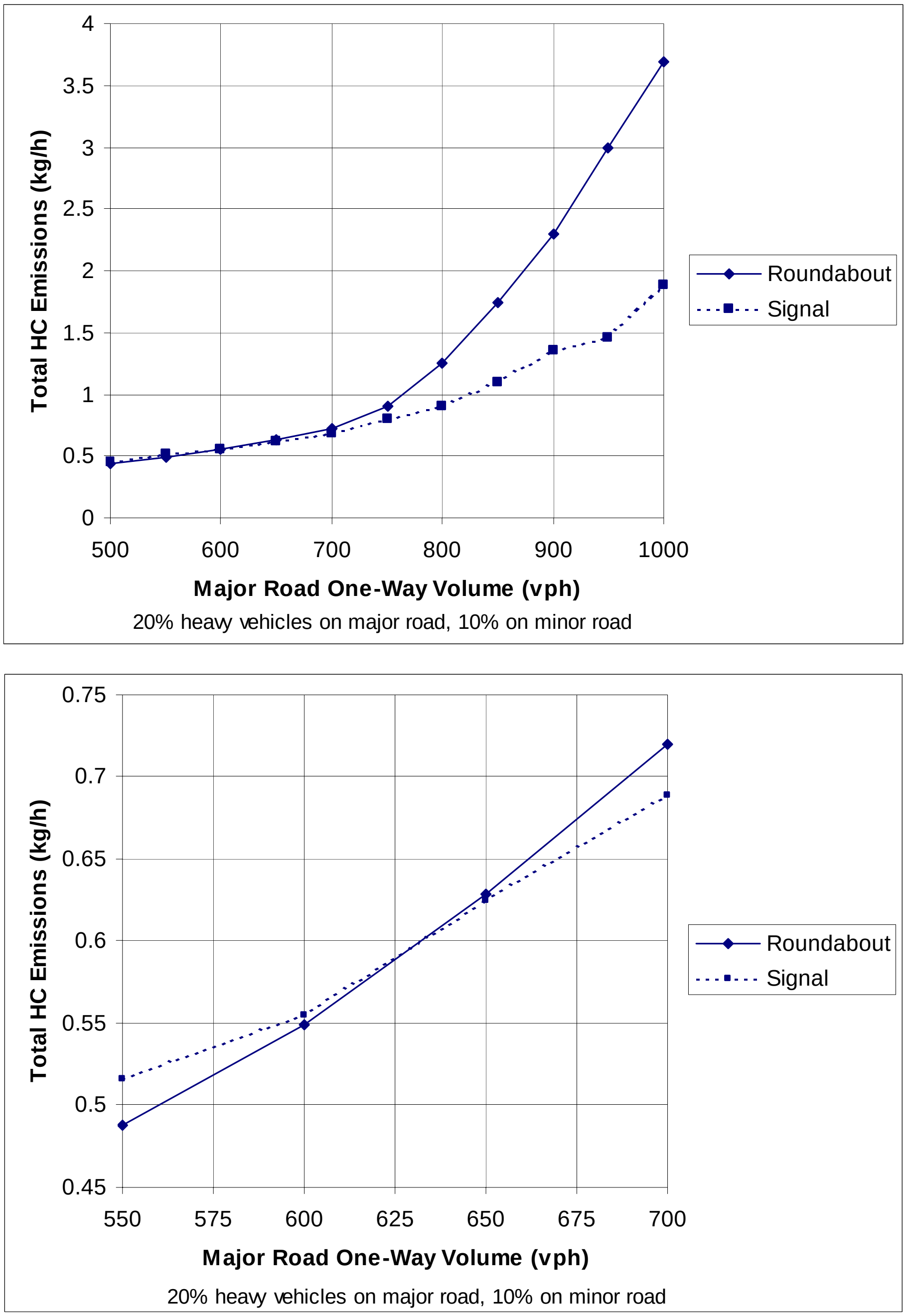
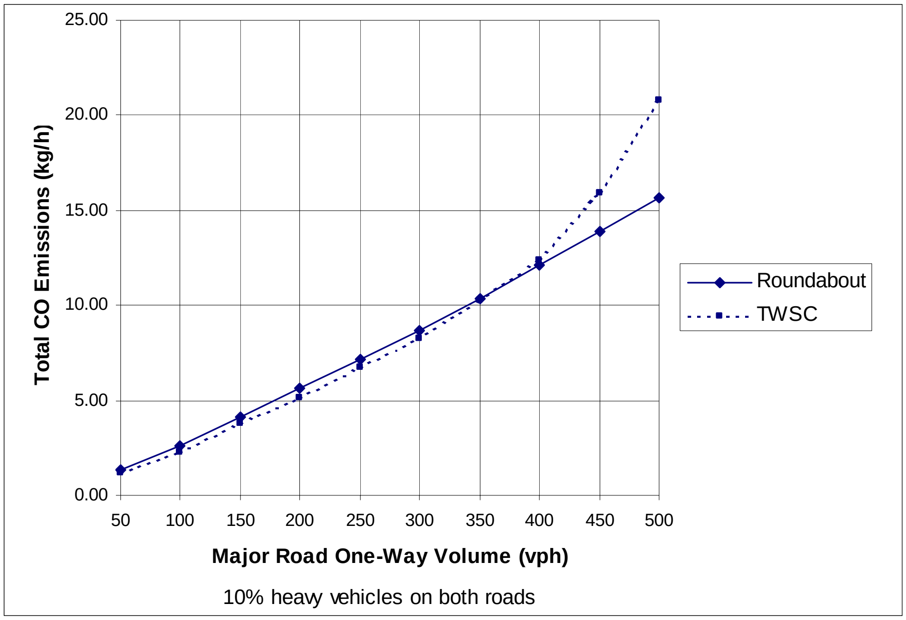


FIGURE 5.19 Total HC Emissions, Roundabout vs. Signalized Intersection

CO Emissions

The CO emissions threshold MRV for a roundabout versus a TWSC intersection, as shown in Figures 5.20 and 5.21, decreases slightly, from MRV 362 to 359 vph as the heavy vehicle percentage is increased on the major road from 10% to 20%.



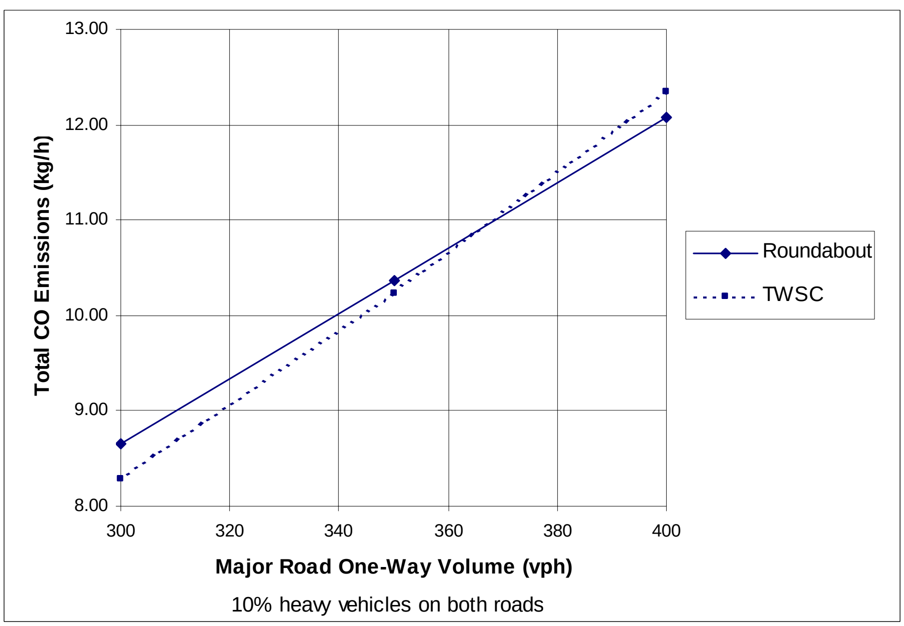


FIGURE 5.20 Total CO Emissions, Roundabout vs. TWSC Intersection

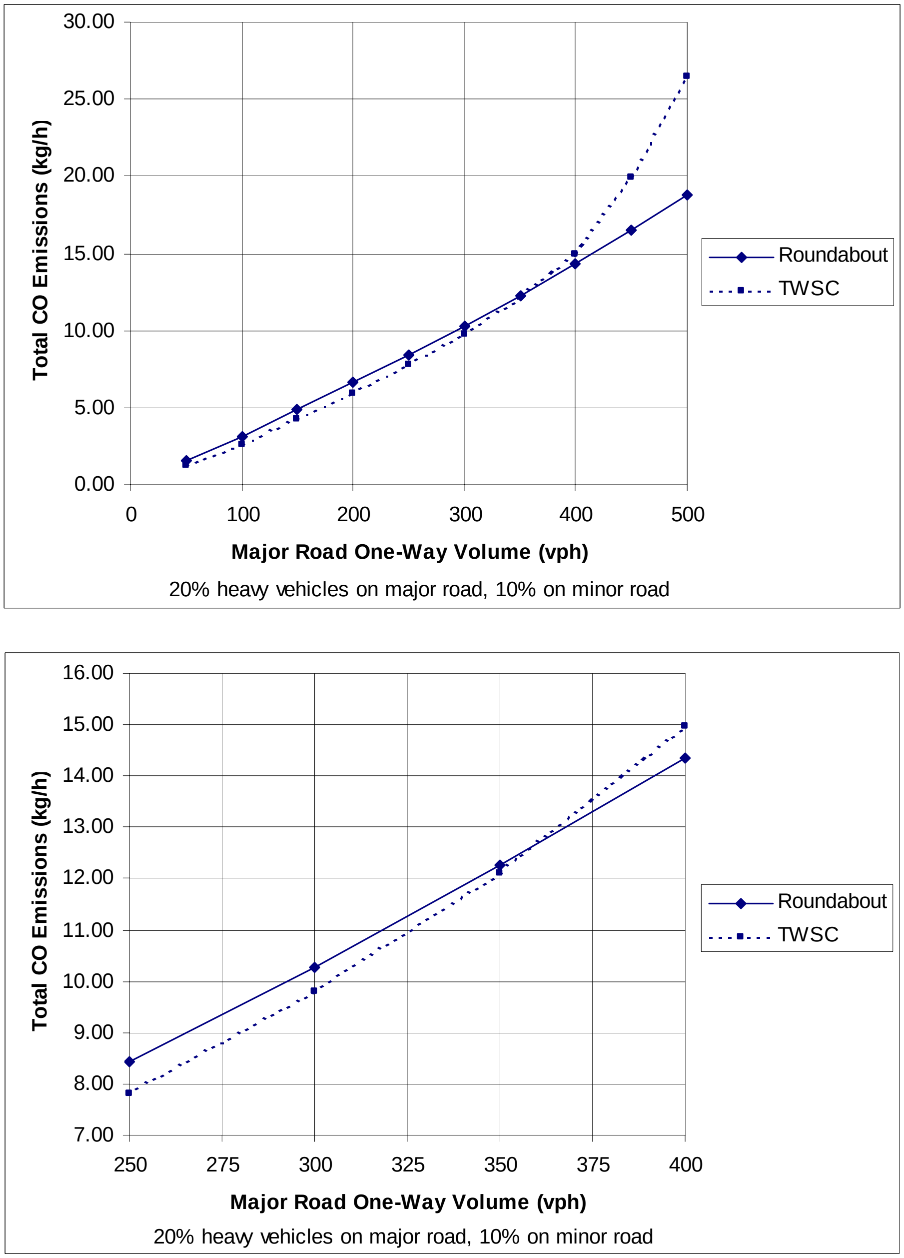


FIGURE 5.21 Total CO Emissions, Roundabout vs. TWSC Intersection

NO_X Emissions

Figure 5.22 shows that for 10% heavy vehicles on both roads, the roundabout provides fewer emissions than a signalized intersection up to an MRV of 592 vph. Figure 5.23 shows that when the heavy vehicle percentage is increased to 20% on the major road, this threshold is reduced to 562 vph. Figure 5.24 presents the same comparison between a roundabout and a TWSC intersection for 10% heavy vehicles on both roads. It can be seen that the TWSC intersection provides fewer NOX emissions than the roundabout below an MRV of 360 vph; after this volume the roundabout performs better in terms of this MOE. Figure 5.25 shows the same comparison after the heavy vehicle percentage on the major road has been increased to 20%; this HV increase results in a small increase of the threshold volume to 367 vph.

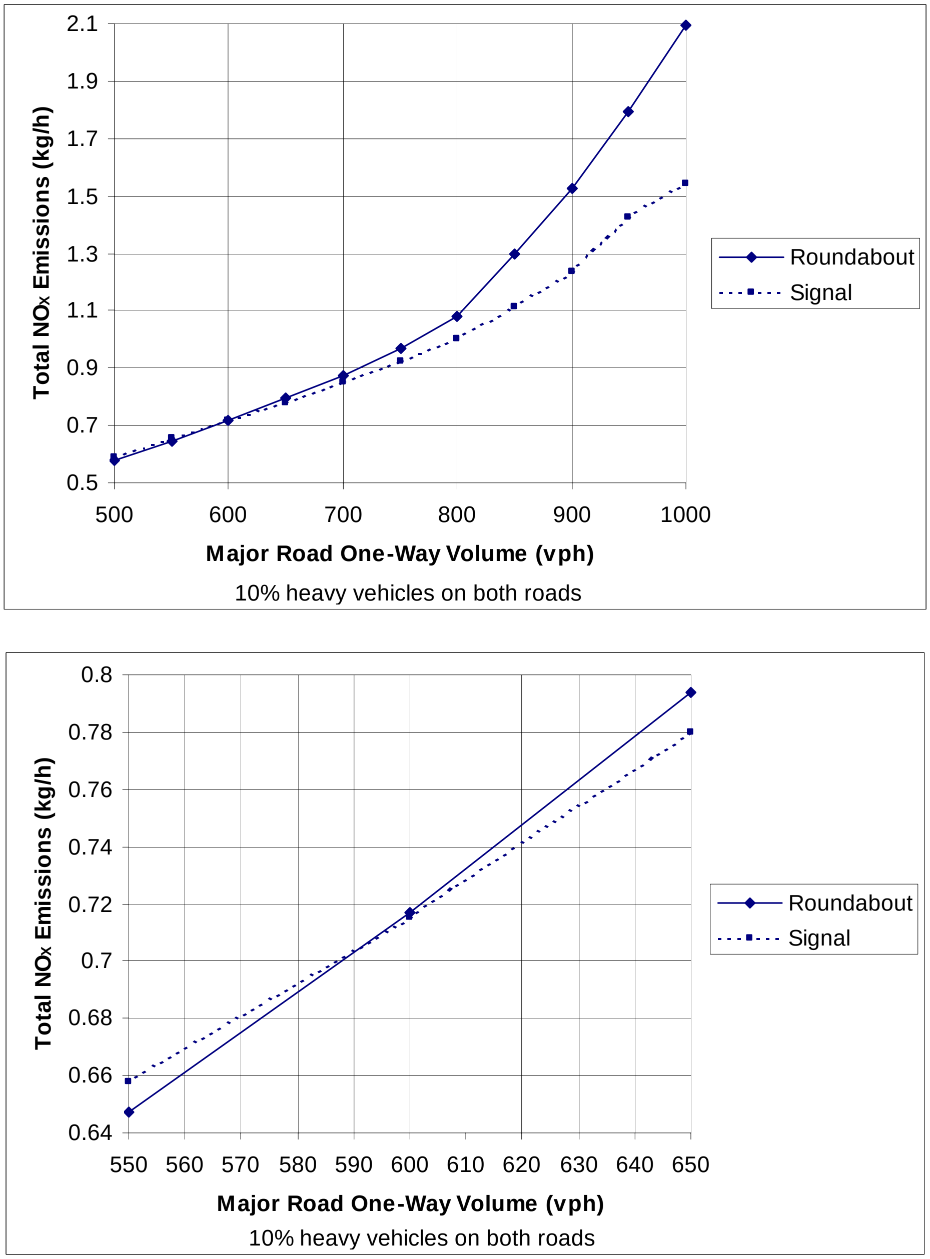
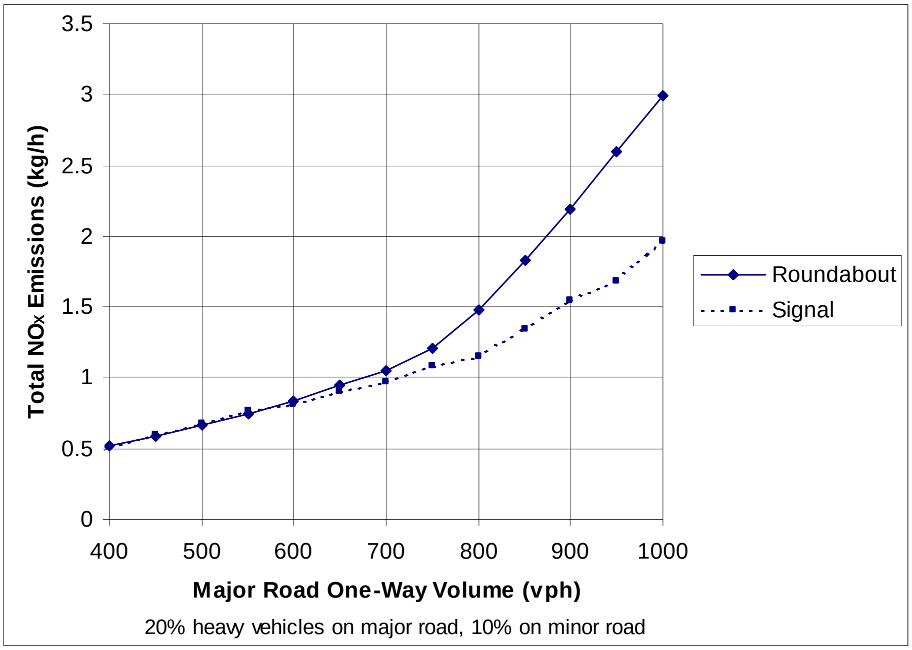


FIGURE 5.22 Total NO_X Emissions, Roundabout vs. Signalized Intersection



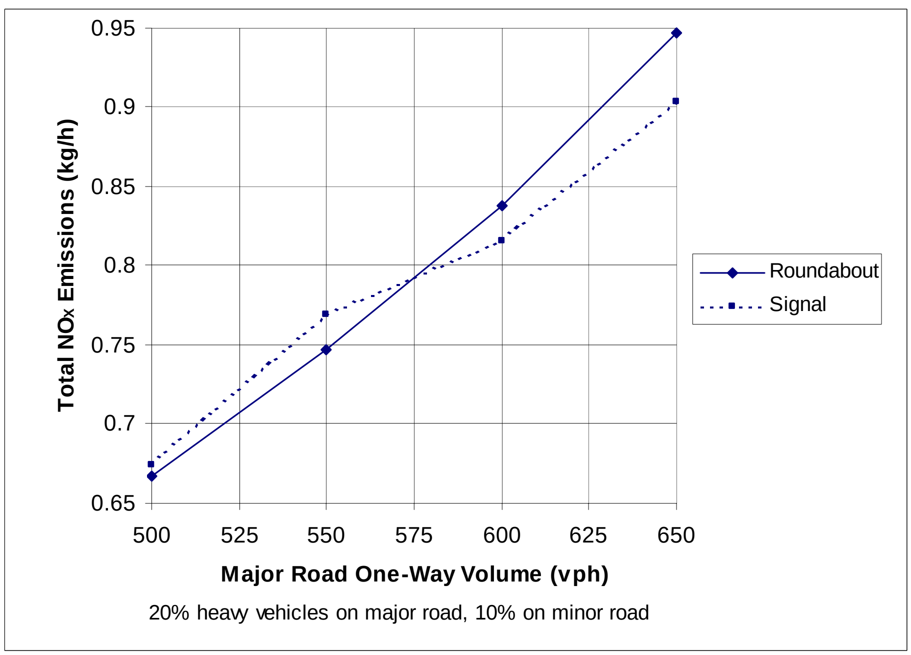


FIGURE 5.23 Total NO_X Emissions, Roundabout vs. Signalized Intersection

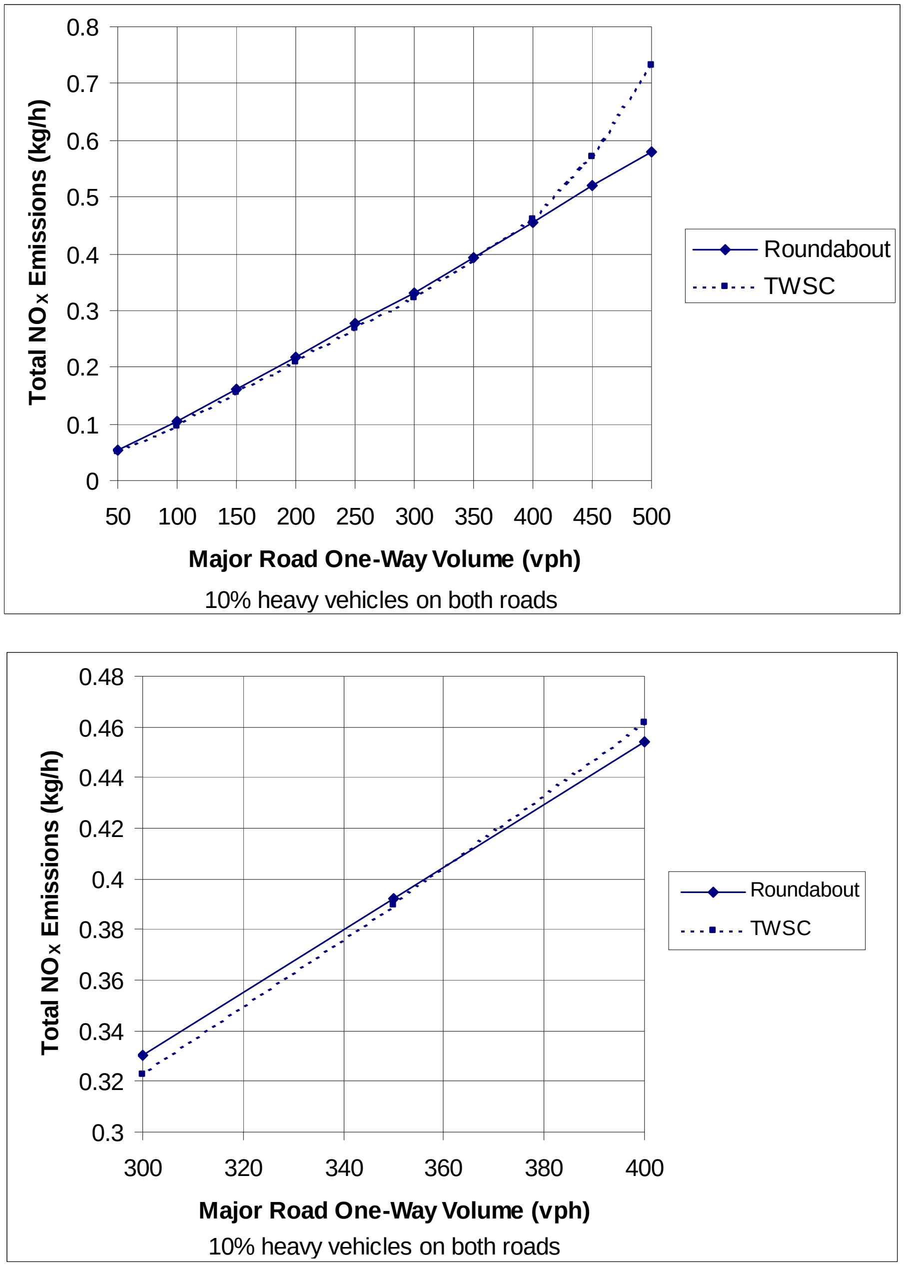


FIGURE 5.24 Total NO_X Emissions, Roundabout vs. TWSC Intersection

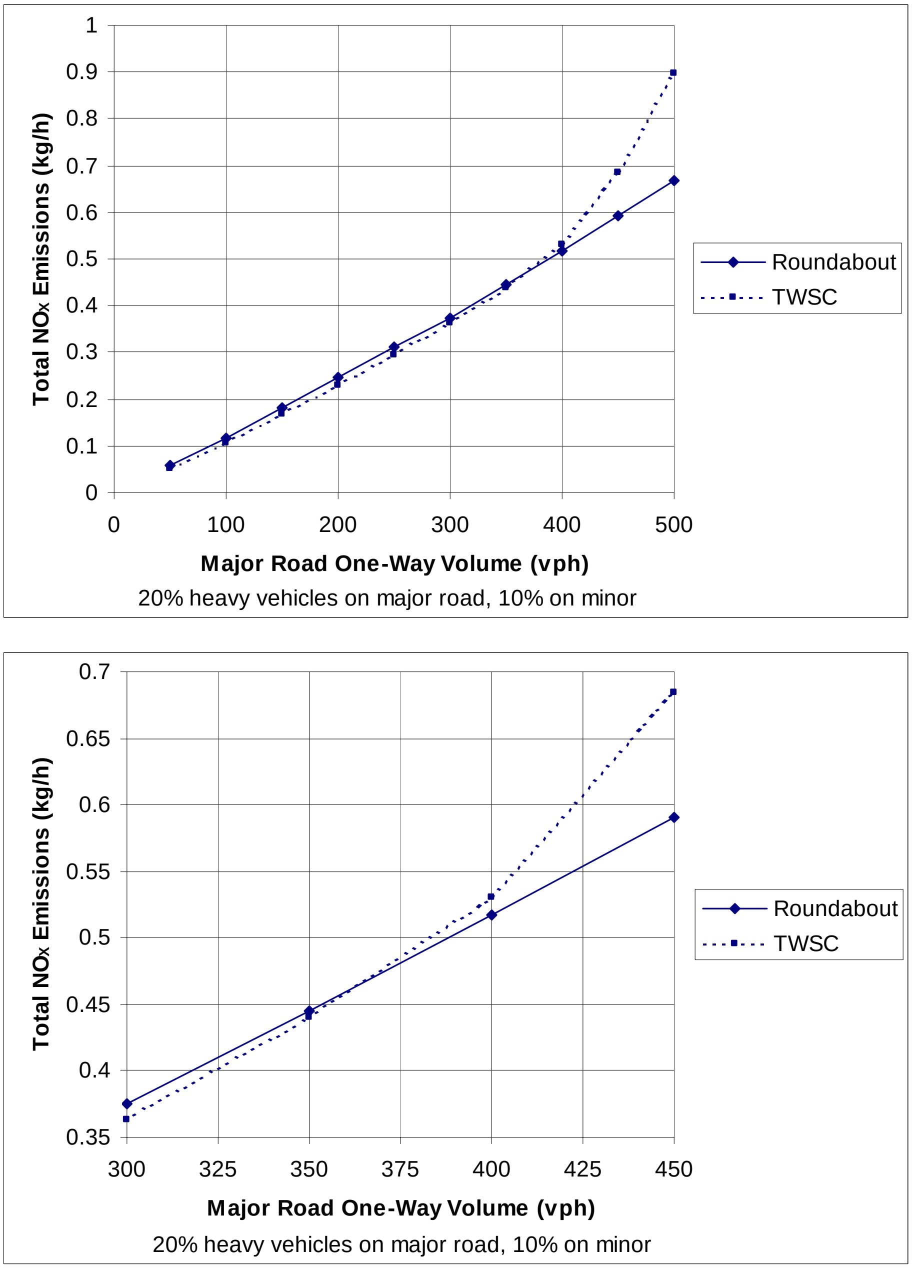


FIGURE 5.25 Total NO_X Emissions, Roundabout vs. TWSC Intersection

CO_2 Emissions

Figure 5.26 shows the comparison between a roundabout and a signalized intersection in terms of CO_2 emissions. It can be seen that for 10% heavy vehicles on both roads, the roundabout provides fewer emissions than the signal below an MRV of 724 vph. Figure 5.27 shows that this threshold volumes decreases to 597 vph with an increase in heavy vehicles on the major road to 20%. A TWSC intersection provides fewer CO_2 emissions than the roundabout up to an MRV of 253 for 10% heavy vehicles on both roads, as presented in Figure 5.28. Figure 5.29 shows that this MRV decreases slightly to 250 vph when the major road heavy vehicle percentage is increased to 20%.

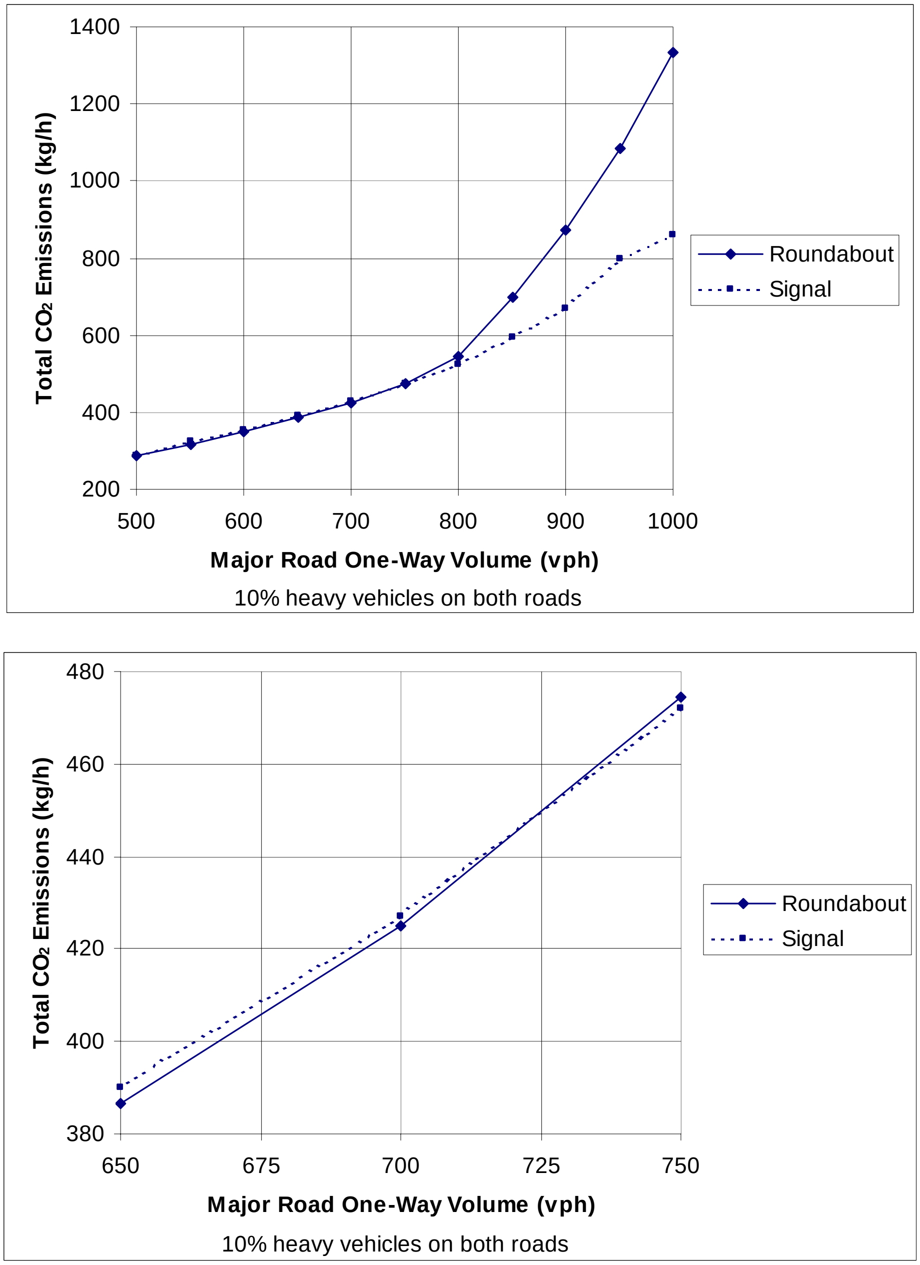
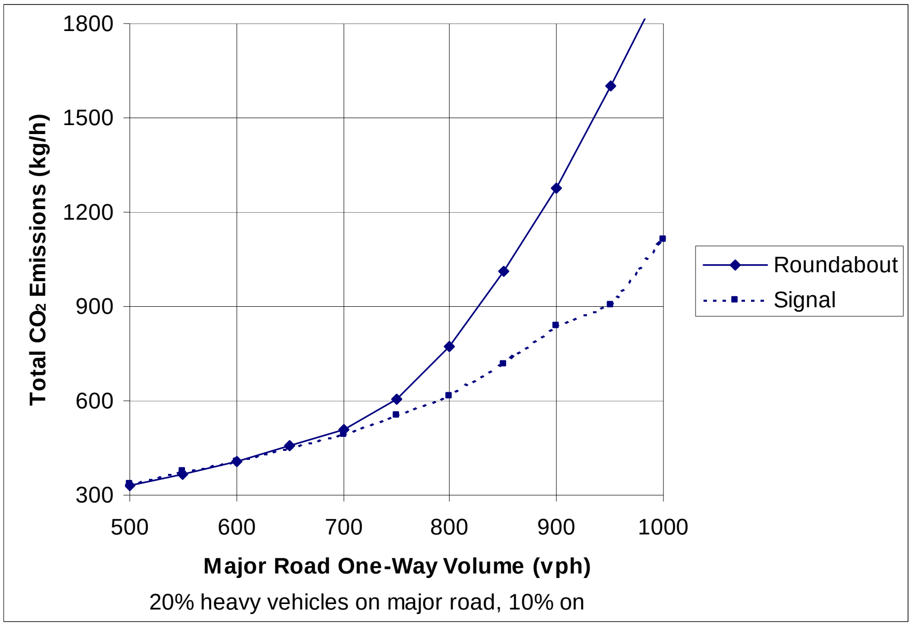


FIGURE 5.26 Total CO_2 Emissions, Roundabout vs. Signalized Intersection



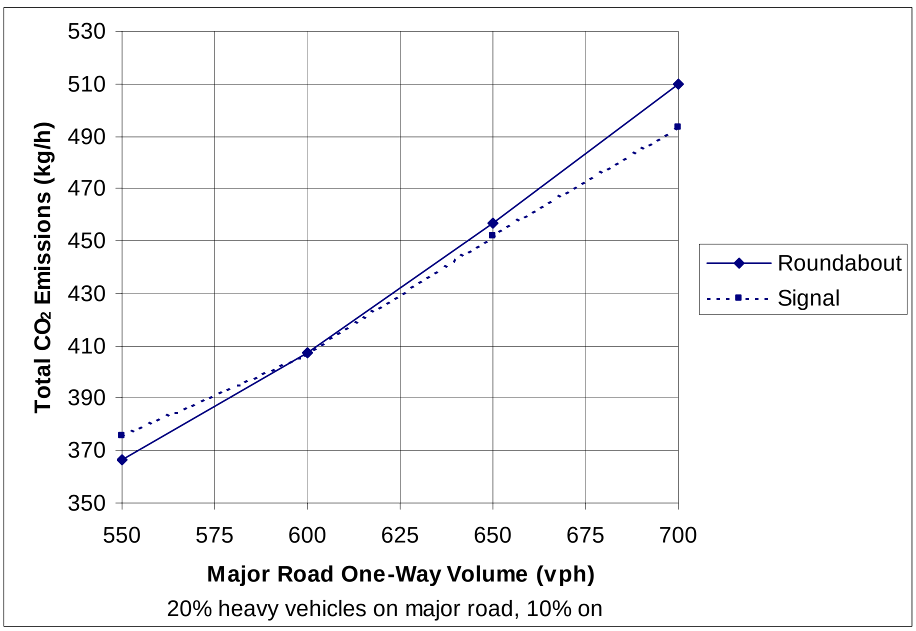


FIGURE 5.27 Total CO_2 Emissions, Roundabout vs. Signalized Intersection

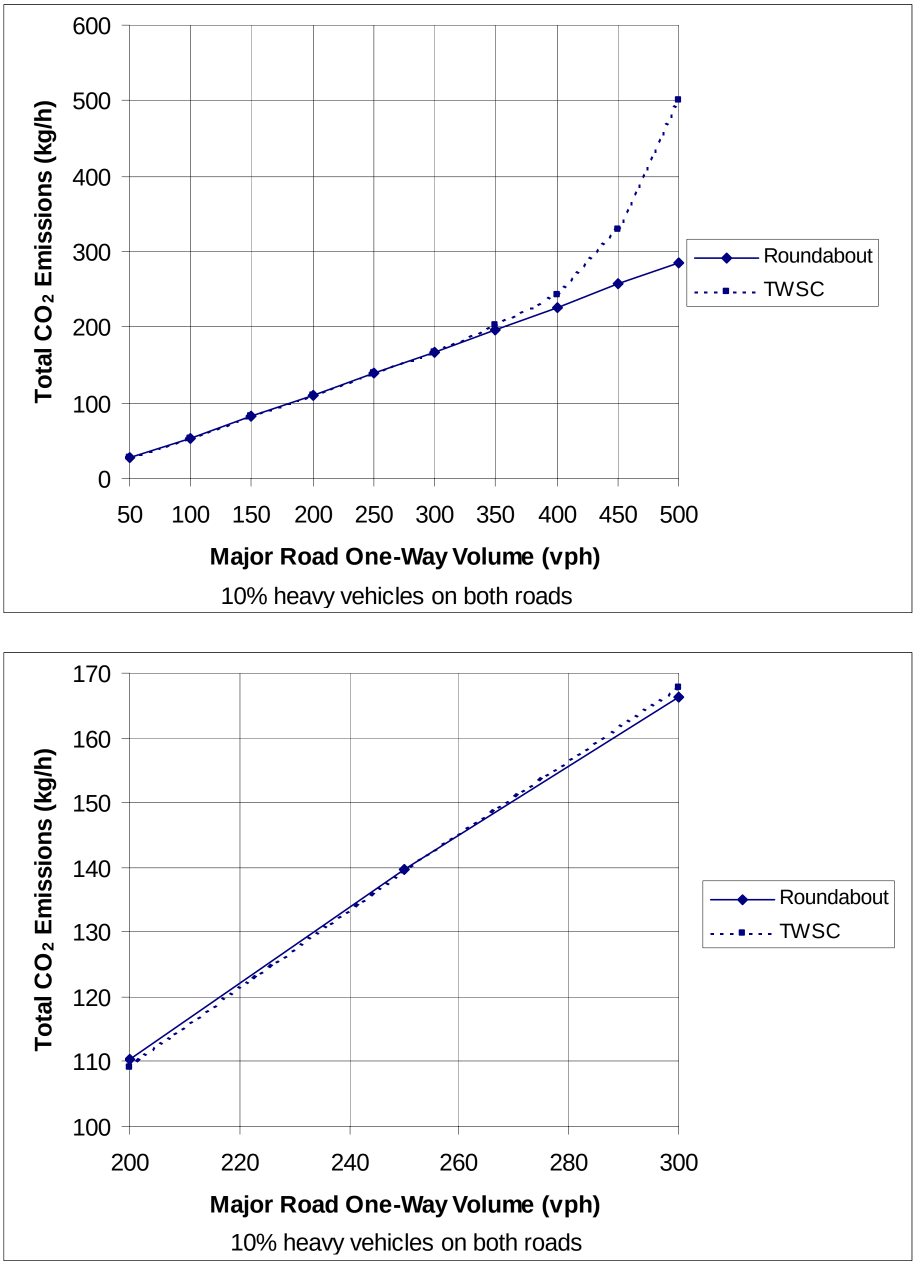
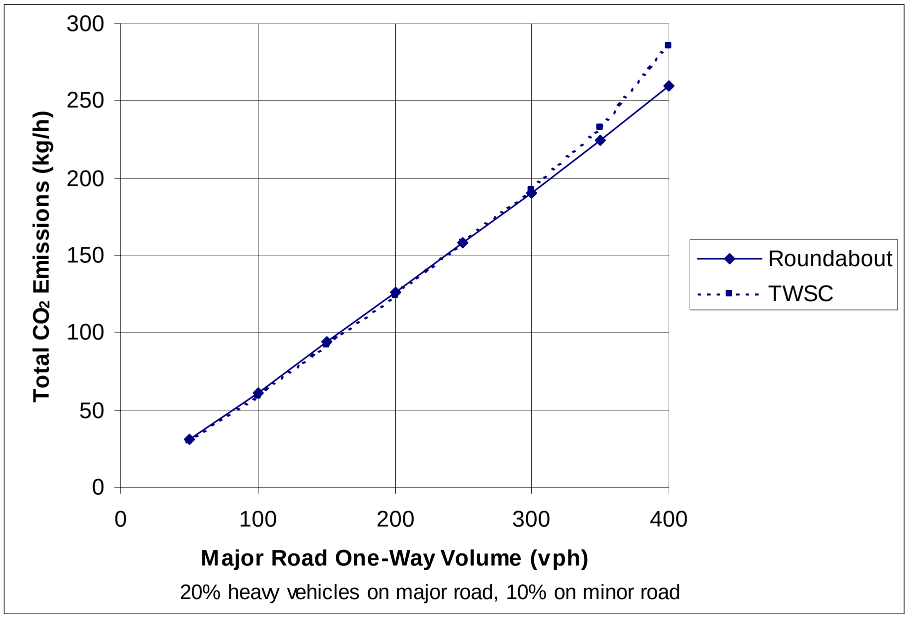


FIGURE 5.28 Total CO_2 Emissions, Roundabout vs. TWSC Intersection



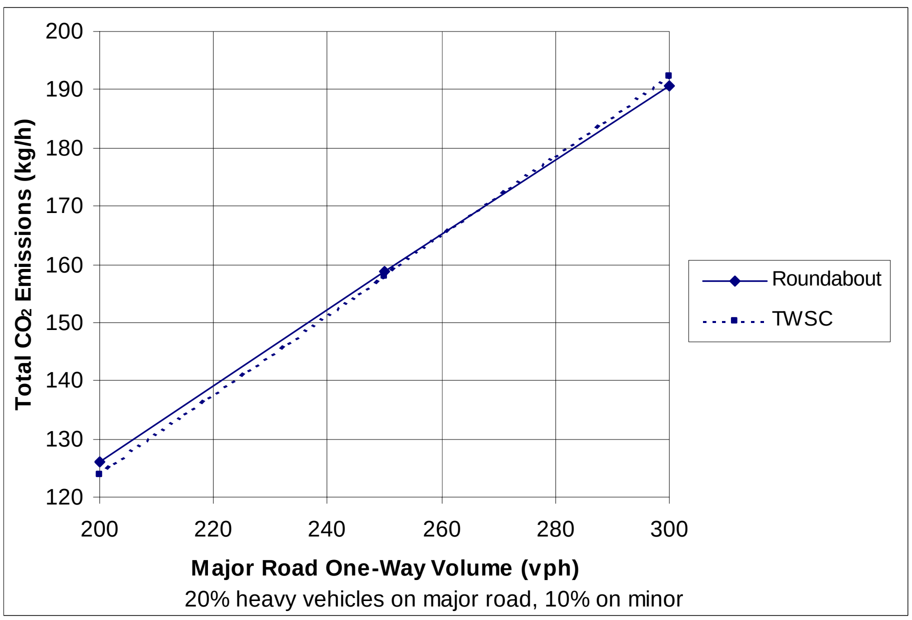


FIGURE 5.29 Total CO_2 Emissions, Roundabout vs. TWSC Intersection

The following tables summarize the major road one-way volume threshold determined for each MOE obtained from the SIDRA analyses using the software's default gap acceptance parameters. Table 5.1 shows threshold values under heavy vehicle conditions of 10% on both roads. Table 5.2 shows threshold values when heavy vehicles have been increased to 20% on the major road while those on the minor road remain at 10%.

TABLE 5.1 Threshold Volumes using SIDRA Default Gap Acceptance Parameters for 10% Heavy Vehicles on Both Roads

	Roundabout vs. TWSC		*Roundabout vs. Signal*	
Measure of Effectiveness	*Advantage*	*Maximum MRV (vph)*	*Advantage*	*Maximum MRV (vph)*
Intersection Capacity (vph)	TWSC	115	Roundabout	680
Major Road Capacity (vph)	N/A	N/A	Roundabout	652
Minor Road Capacity (vph)	TWSC	440	Roundabout	653
Major Road Delay (s)	N/A	N/A	Roundabout	628
Minor Road Delay (s)	Roundabout	**	Roundabout	830
Major Road 95% Queue (ft)	N/A	N/A	Roundabout	844
Minor Road 95% Queue (ft)	Roundabout	**	Roundabout	929
Total HC Emissions (kg/h)	TWSC	250	Roundabout	750
Total CO Emissions (kg/h)	TWSC	362	Roundabout	613
Total NO_X Emissions (kg/h)	TWSC	360	Roundabout	592
Total CO_2 Emissions (kg/h)	TWSC	253	Roundabout	724

* Major One-Way Road Volume up to which the indicated intersection type shows better performance
** Intersection type shows better performance throughout the entire range of volumes considered

TABLE 5.2 Threshold Volumes using SIDRA Default Gap Acceptance Parameters for 20% Heavy Vehicles on the Major Road, 10% on the Minor Road

	Roundabout vs. TWSC		*Roundabout vs. Signal*	
Measure of Effectiveness	*Advantage*	*Maximum MRV (vph)*	*Advantage*	*Maximum MRV (vph)*
Intersection Capacity (vph)	TWSC	114	Roundabout	623
Major Road Capacity (vph)	N/A	N/A	Roundabout	618
Minor Road Capacity (vph)	Roundabout	**	Roundabout	728
Major Road Delay (s)	N/A	N/A	Roundabout	583
Minor Road Delay (s)	Roundabout	**	Roundabout	944
Major Road 95% Queue (ft)	N/A	N/A	Roundabout	694
Minor Road 95% Queue (ft)	Roundabout	**	Roundabout	987
Total HC Emissions (kg/h)	TWSC	270	Roundabout	630
Total CO Emissions (kg/h)	TWSC	359	Roundabout	586
Total NO_X Emissions (kg/h)	TWSC	367	Roundabout	562
Total CO_2 Emissions (kg/h)	TWSC	258	Roundabout	597

* Major One-Way Road Volume up to which the indicated intersection type shows better performance
** Intersection type shows better performance throughout the entire range of volumes considered

6.0 FINAL ANALYSIS USING MARYLAND GAP ACCEPTANCE VALUES

Following data reduction, a second set of analyses was conducted using the SIDRA software to compare the three intersection types. This set of analyses followed the same procedures as the first in all but the gap acceptance parameters, which were determined from the videotapes to be a critical gap of 3.9 s and a follow-up time of 2.0 s. Measures of effectiveness consists of effective intersection capacity, major road capacity, minor road capacity, major road average delay, minor road average delay, major road 95% queue length, minor road 95% queue length, and CO, CO_2, NO_X, and HC emission rates. As in the preliminary analysis, two heavy vehicle conditions were tested, the first with 10% heavy vehicles on both roads, and the second with 20% heavy vehicles on the main road and 10% heavy vehicles on the minor road.

6.1 CAPACITY

According to earlier tests performed in the preliminary SIDRA analysis, it was clear that the relevant comparisons were roundabouts versus unsignalized intersections at lower volumes and roundabouts versus signalized intersections at higher volumes.

Effective intersection capacity

Figure 6.1 shows that the roundabout provides greater effective intersection capacity than the TWSC intersection throughout the range of volumes tested in both heavy vehicle situations. With increasing MRV, the roundabout capacity steadily decreased while that of the TWSC intersection decreased at a greater rate. The capacities of both intersection types decreased slightly as the heavy vehicle percentage increased. Figure 6.2 shows that prior to an MRV of 809 vph, the roundabout provides greater effective intersection capacity than the signalized intersection. This crossover point is reduced to 761 vph when the heavy vehicle percentage is increased.

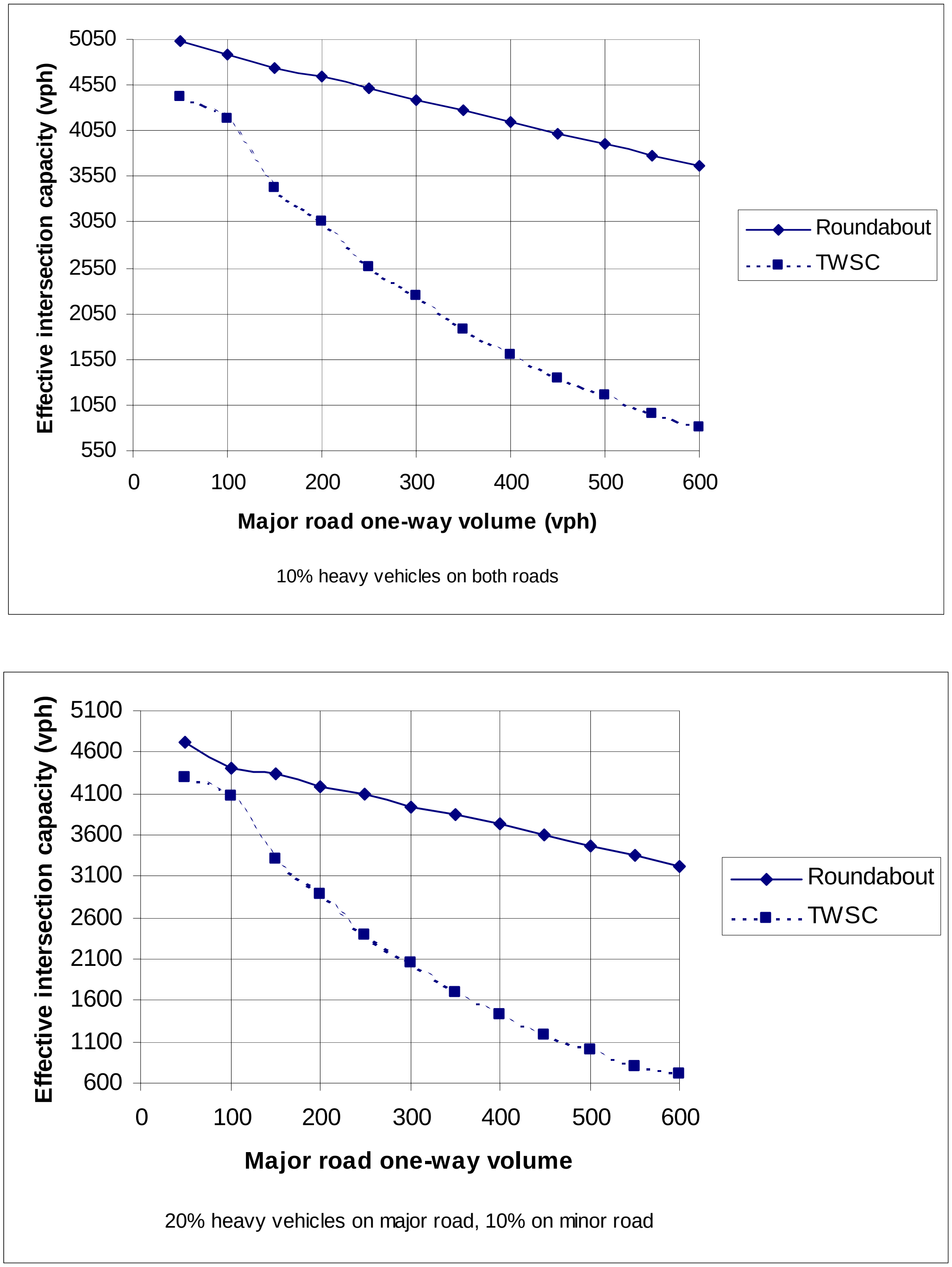


FIGURE 6.1 Effective Intersection Capacity, Roundabout vs. TWSC Intersection

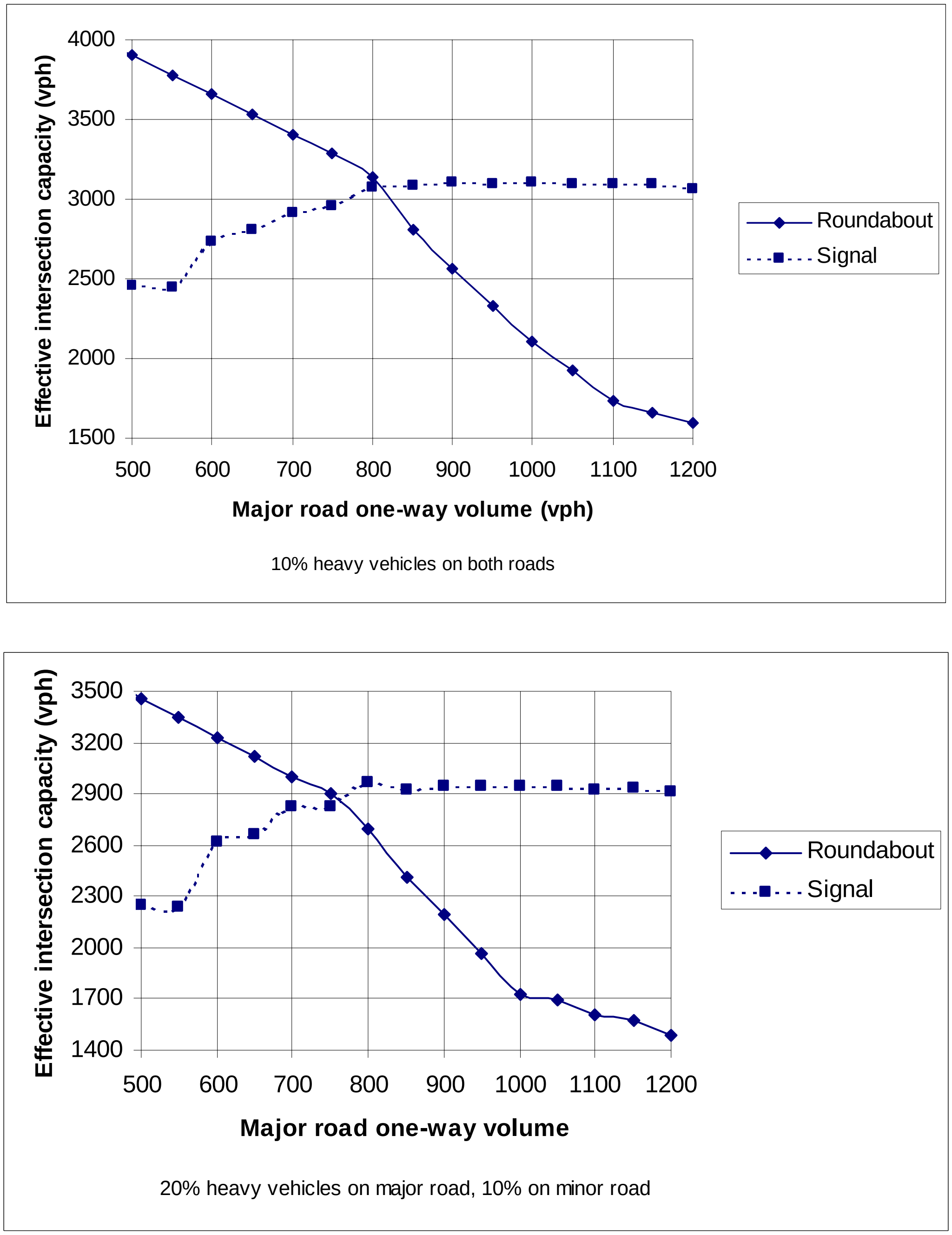
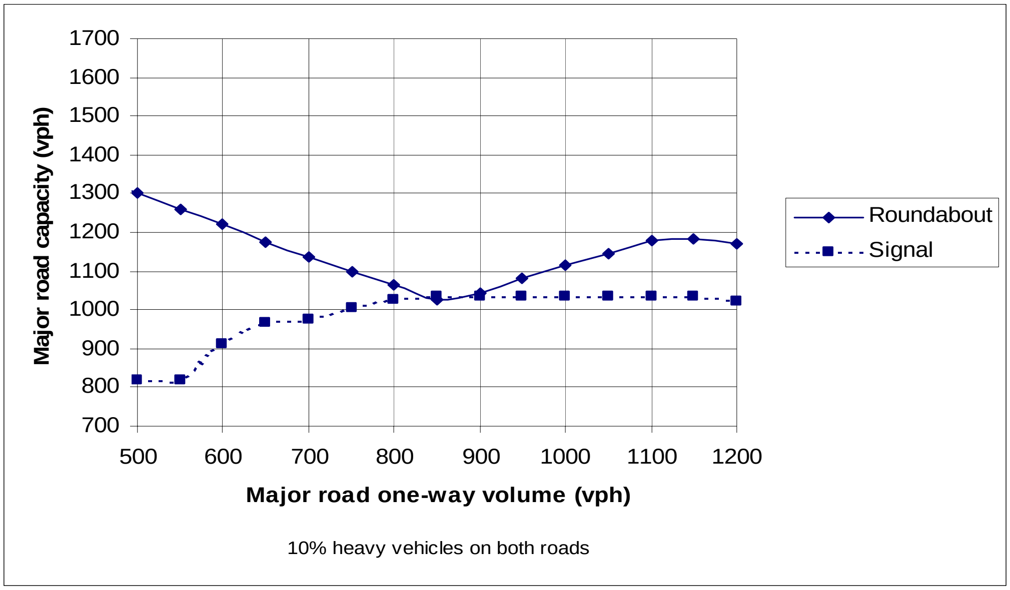


FIGURE 6.2 Effective Intersection Capacity, Roundabout vs. Signalized Intersection

Major road capacity

In the case of roundabout versus TWSC intersection, it was decided that comparisons based upon the major road (capacity, delay, queue) were irrelevant due to the fact that the major road in a TWSC intersection is always given priority and therefore will always provide greater capacity, less delay, and a smaller queue than the minor road. The three major road MOEs were therefore not included in the analysis of roundabout versus TWSC intersection. The major road capacity for the roundabout versus signalized intersection is shown in Figure 6.3. The roundabout provides greater major road capacity than the signalized intersection throughout the range of volumes considered except at an MRV of 850 vph, where the signalized intersection provides slightly greater capacity (1034 vph vs. 1025 vph for the roundabout).



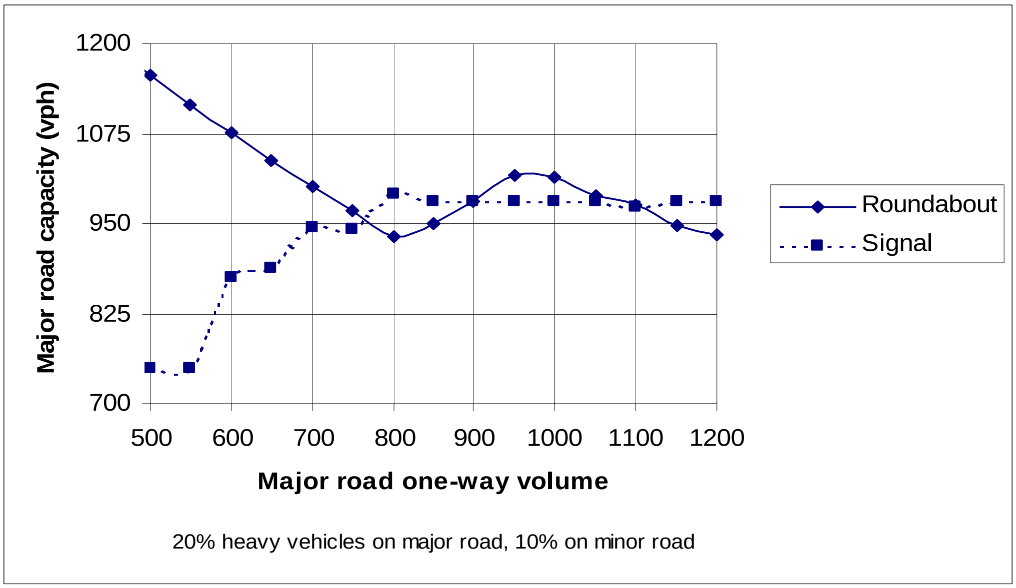
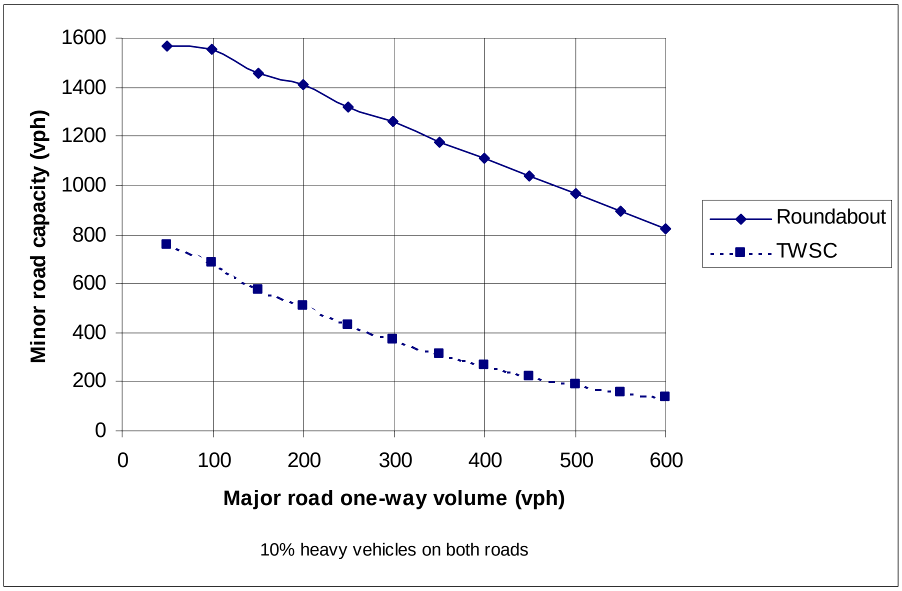


FIGURE 6.3 Major Road Capacity, Roundabout vs. Signalized Intersection

Minor road capacity

The minor road capacity is shown for roundabout versus TWSC intersection in Figure 6.4. It can be seen that for both heavy vehicle conditions the roundabout provides the greater capacity throughout the range of MRV tested. The capacity of both intersection types remained essentially the same as the heavy vehicle percentage was increased. Figure 6.5 shows the minor road capacity for roundabout versus signalized intersection. The graphs show that up until an MRV of 805 vph, the roundabout provides greater minor road capacity than the signal. After this point, however, the signal provides the greater capacity. This threshold volume decreases to 766 vph when the heavy vehicle percentage was increased.



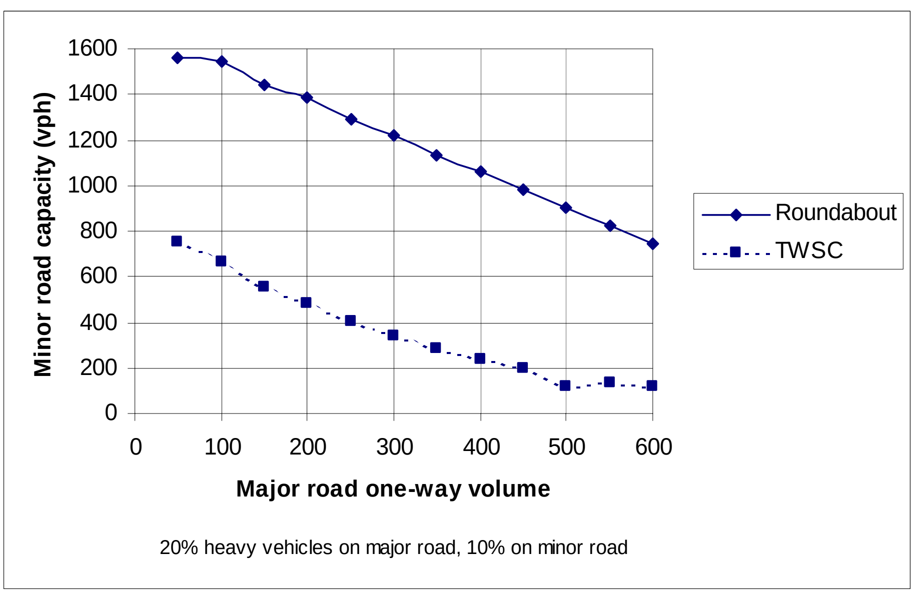
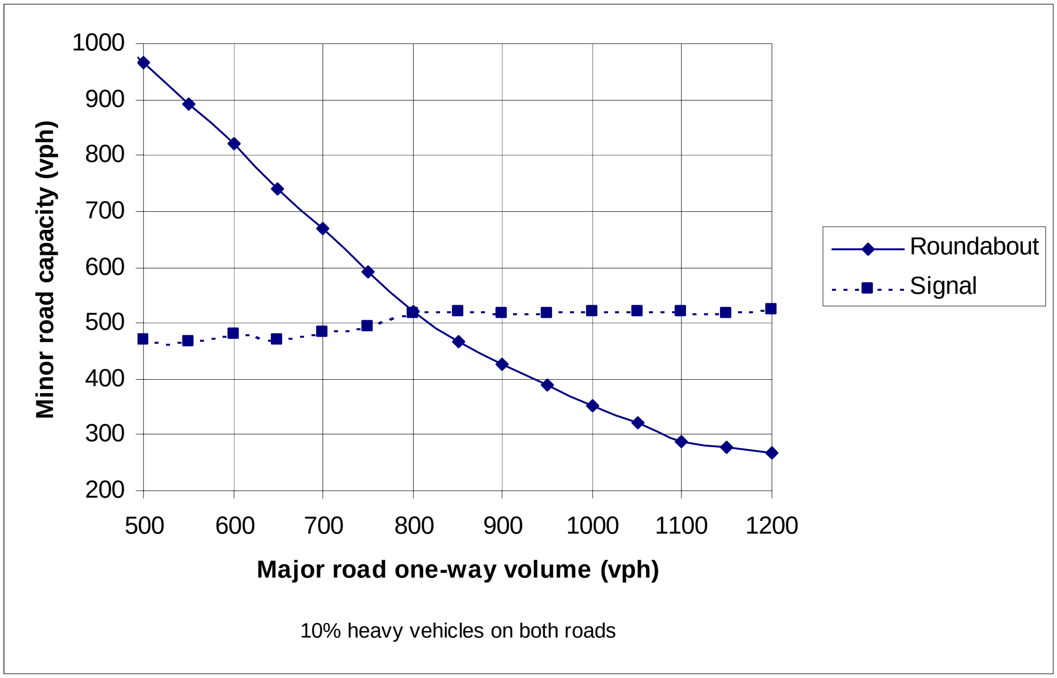


FIGURE 6.4 Minor Road Capacity, Roundabout vs. TWSC Intersection



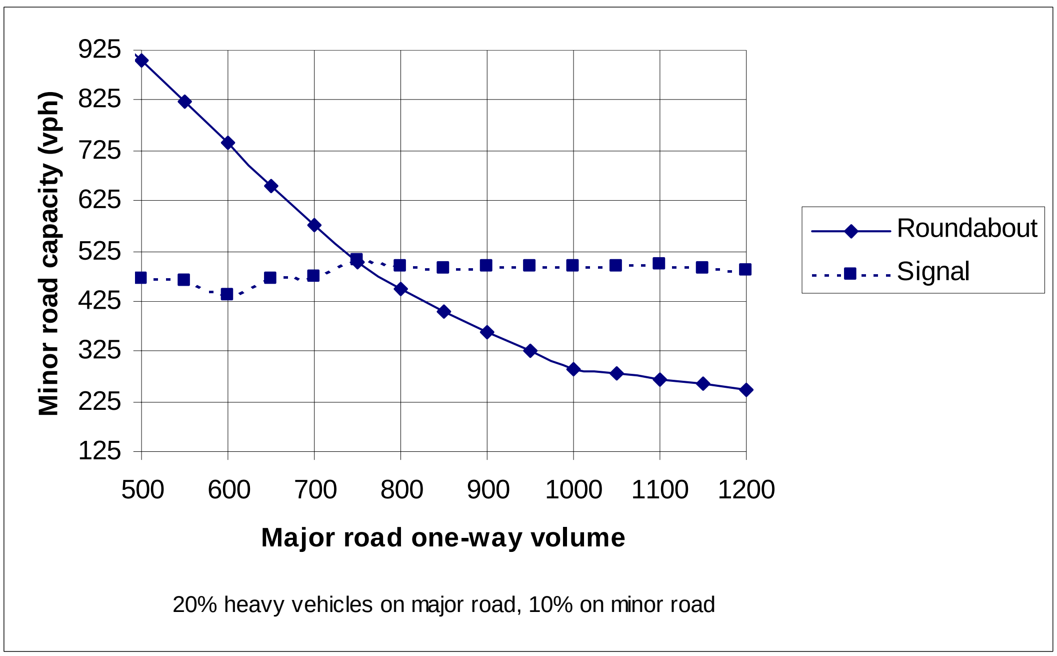
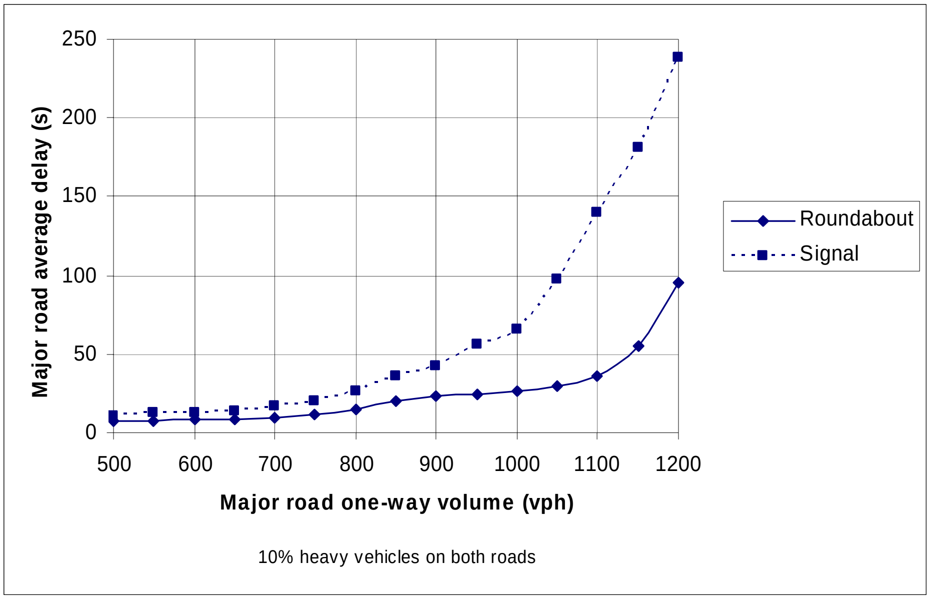


FIGURE 6.5 Minor Road Capacity, Roundabout vs. Signalized Intersection

6.2 DELAY

Major road average delay

This MOE was not included for the comparison of roundabout versus TWSC intersection, as explained above. Figure 6.6 shows the comparison of roundabout versus signalized intersection. The roundabout provides less delay throughout the range of MRV considered for 10% heavy vehicles exist on both roads. When this percentage is increased along the major road, a threshold volume occurs at 1111 vph, where after this point the signal provides less delay than the roundabout.



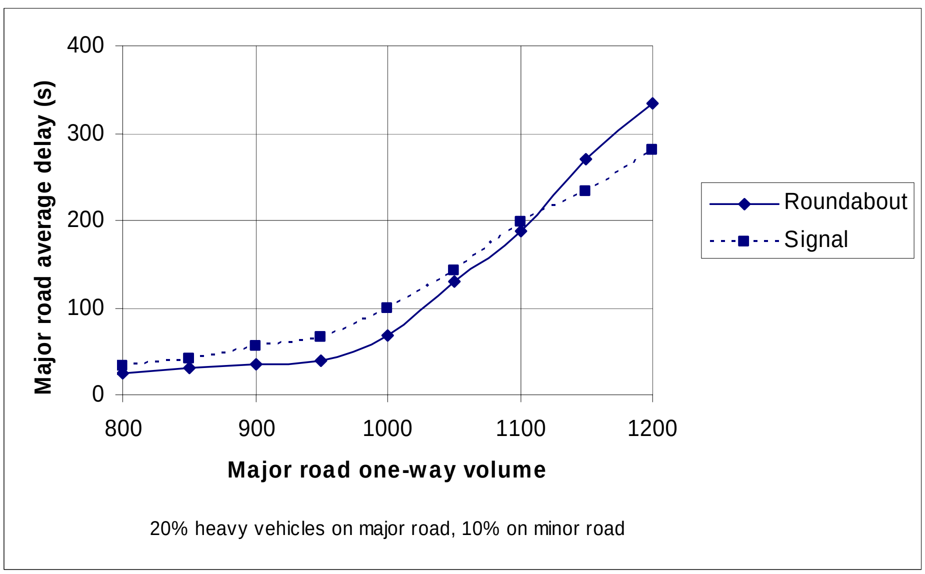


FIGURE 6.6 Major Road Average Delay, Roundabout vs. Signalized Intersection

Minor road average delay

Figure 6.7 shows the minor road average delay for roundabout versus TWSC intersection. Throughout the range of MRV considered and for both heavy vehicle situations, the roundabout provides less delay. Prior to an MRV of 400 vph, both intersection types provide relatively similar delay; after this point, however, the delay associated with the TWSC intersection increases exponentially, while that for the roundabout continues to gradually increase at a steady rate. Figure 6.8 presents the minor road average delay for roundabout versus signalized intersection. The roundabout provides less delay up until an MRV of 833 vph for 10% HV on both roads and up until 802 vph for 20% HV on the major road. After these threshold volumes, delay associated with the roundabout increases exponentially.

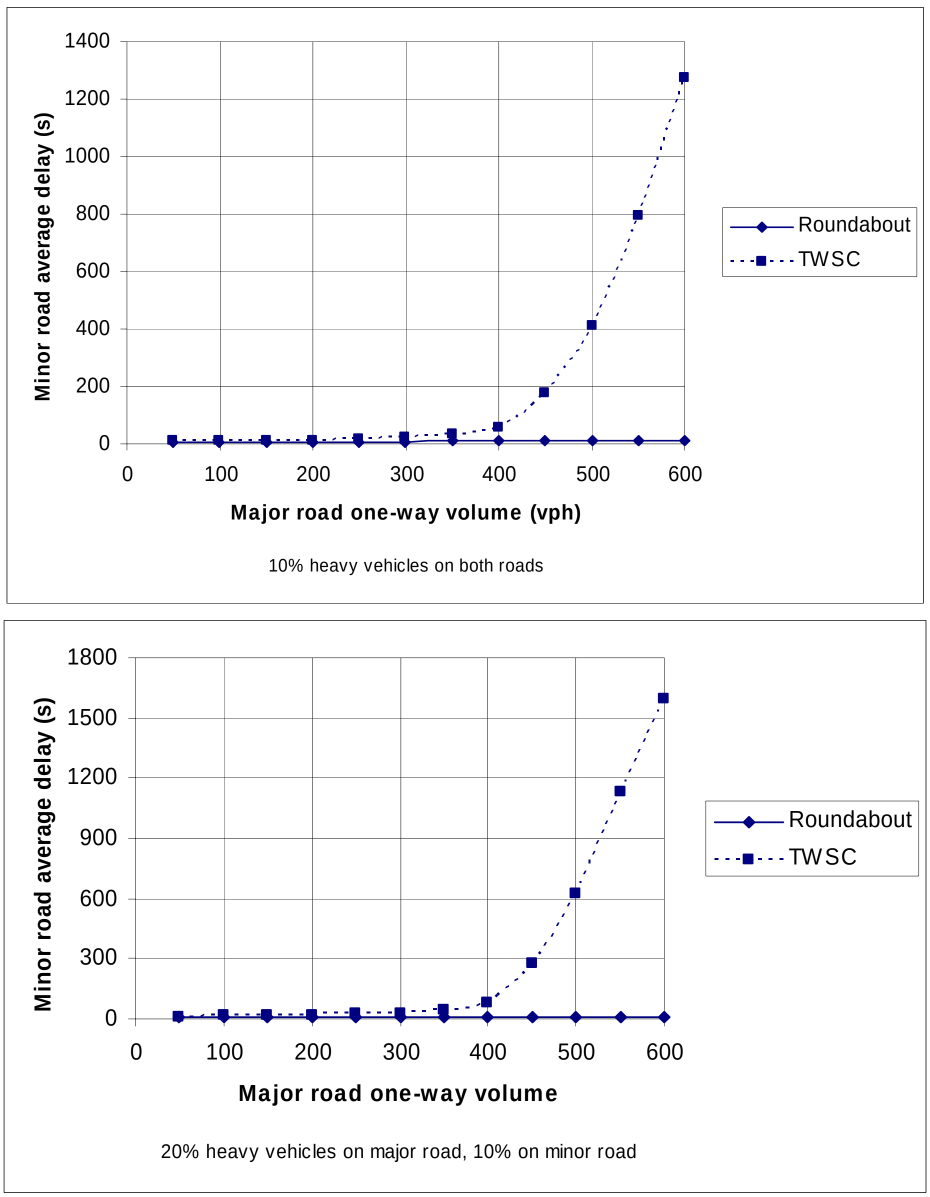
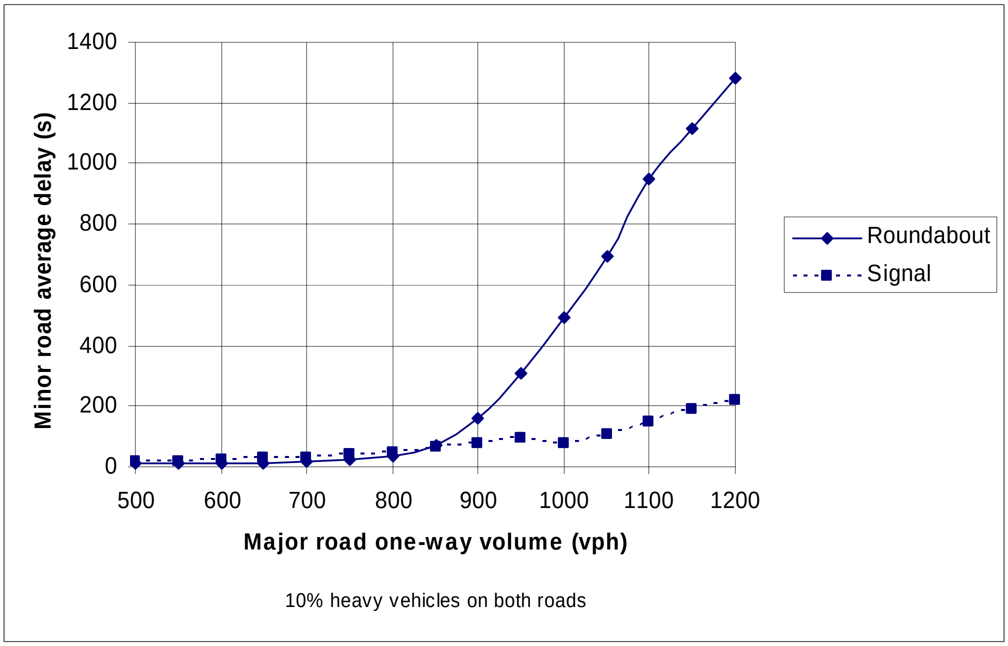


FIGURE 6.7 Minor Road Average Delay, Roundabout vs. TWSC Intersection



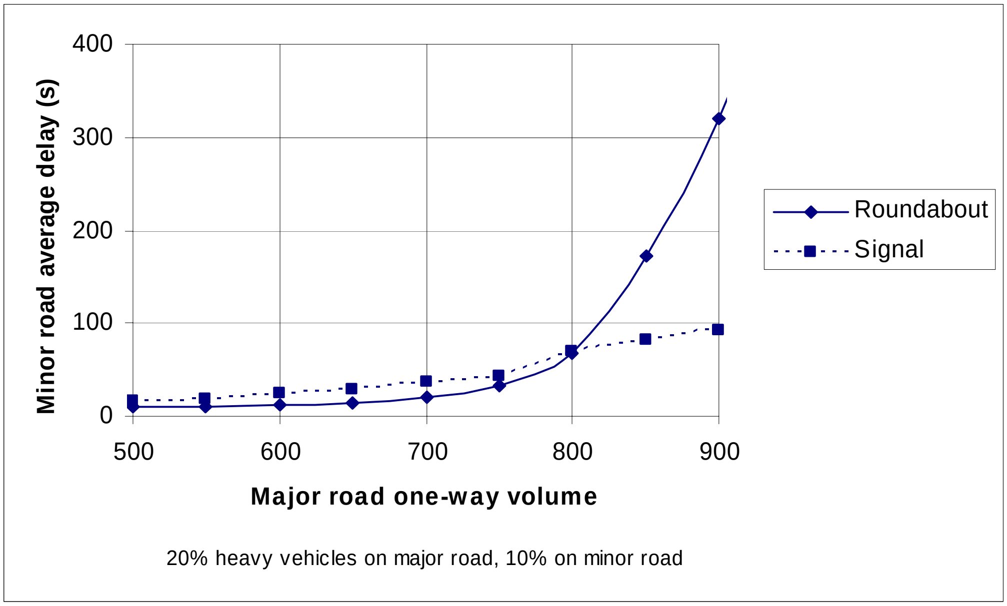


FIGURE 6.8 Minor Road Average Delay, Roundabout vs. Signalized Intersection

6.3 QUEUE LENGTH

<u>Major road 95% queue</u>

This MOE was not included in the comparison of roundabout versus TWSC intersection, for reasons explained above. Figure 6.9 shows this MOE for the comparison of roundabout versus signalized intersection. It can be seen that the roundabout provides a shorter queue length than the signal when there is 10% HV on both roads. When the HV percentage is increased, a threshold volume occurs at an MRV of 1114 vph, where after this point, the roundabout begins to provide a greater queue length than the signal.

<u>Minor road 95% queue</u>

Figure 6.10 presents the minor road queue length for roundabout versus TWSC intersection. For both HV percentage situations, the roundabout provides a shorter queue length throughout the range of MRV tested. The TWSC intersection queue begins to rapidly increase after an MRV of 400 vph while that for the roundabout increases at a low and steady rate. Figure 6.11 shows this MOE for roundabout versus signalized intersection. This figure shows that the roundabout provides a smaller queue up to an MRV of 871 vph for 10% HV on both roads, and up to 817 vph when the heavy vehicles are increased on the major road. Prior to these threshold volumes, the queues for both intersection types are relatively similar, whereas after the thresholds, the difference between the two greatly increases.

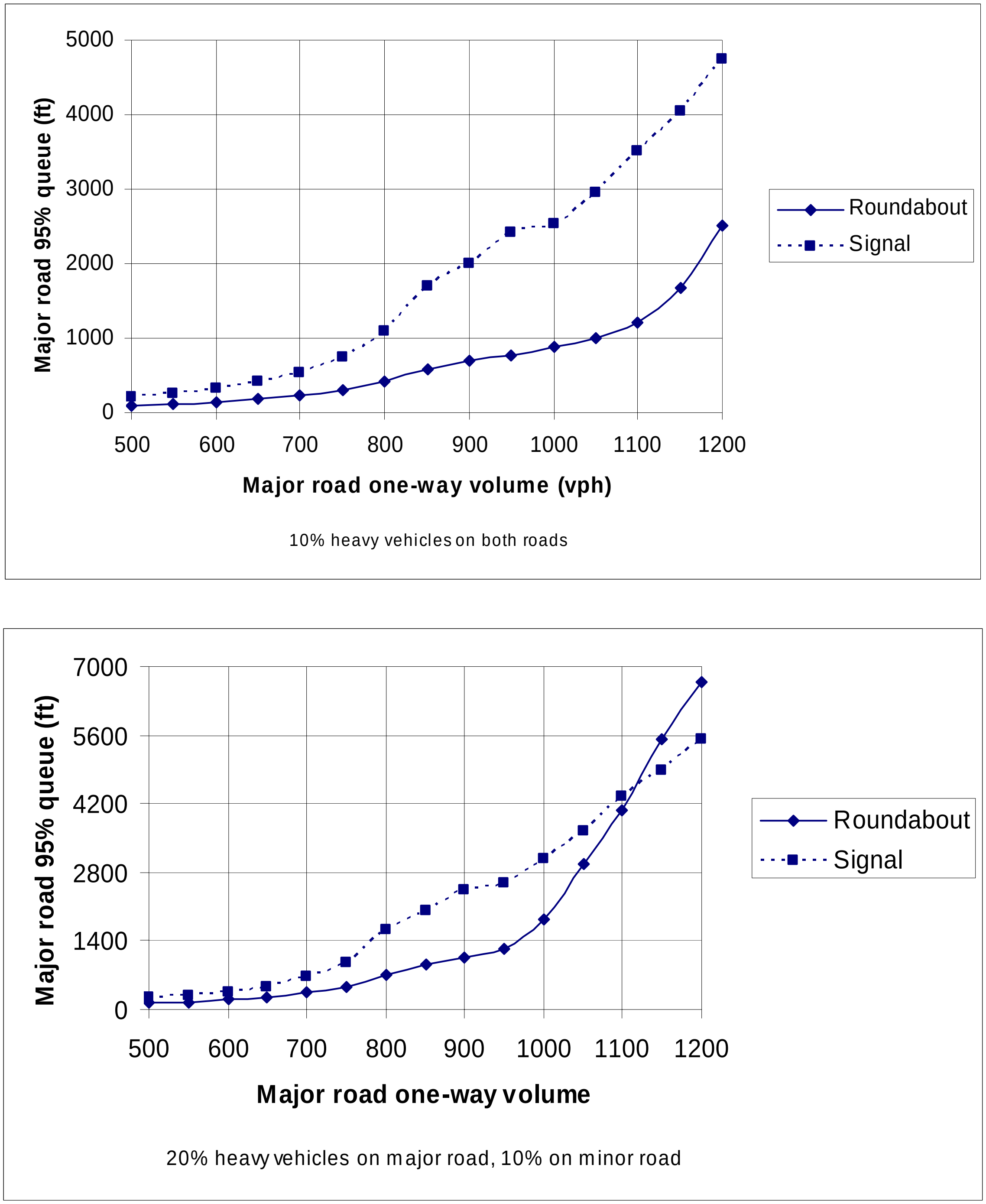
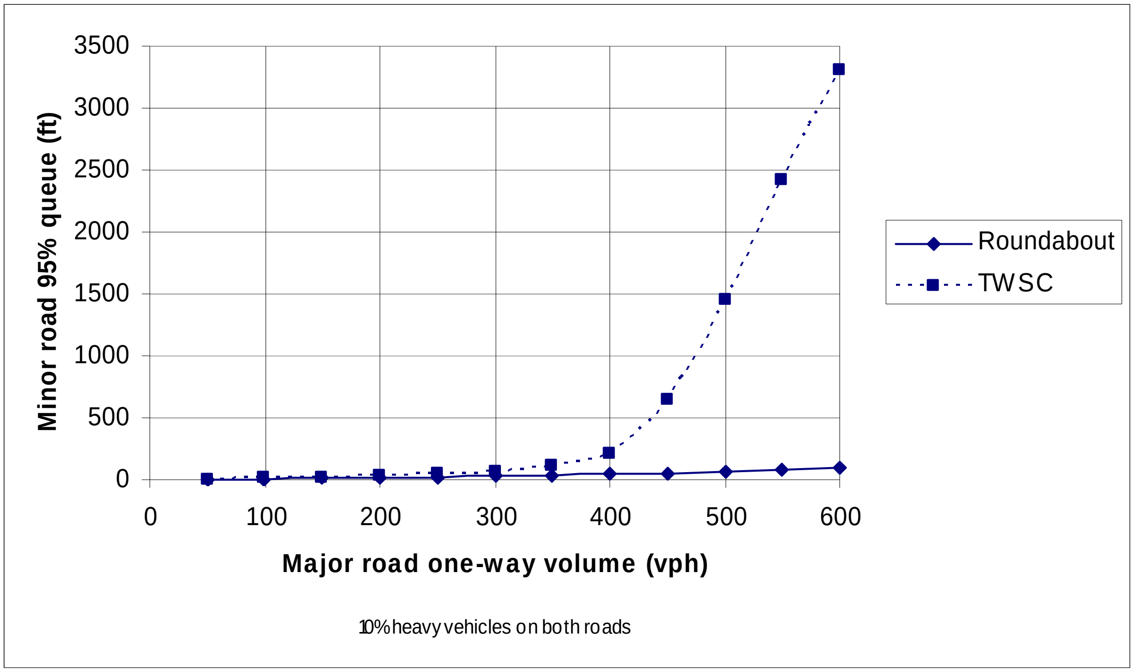


FIGURE 6.9 Major Road Queue, Roundabout vs. Signalized Intersection



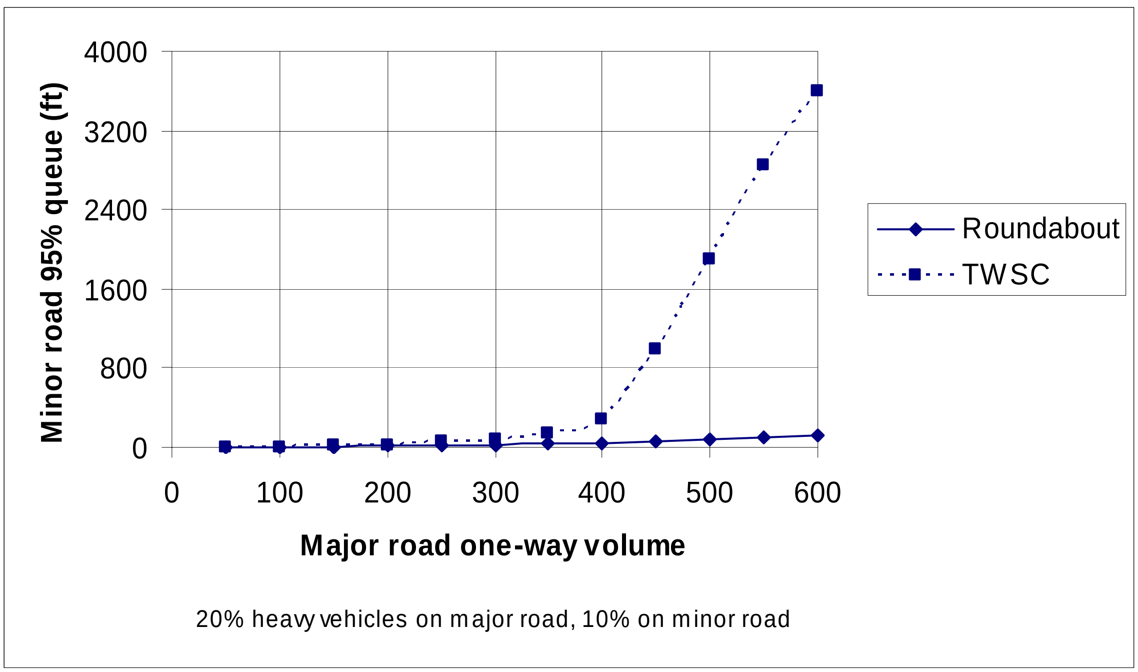
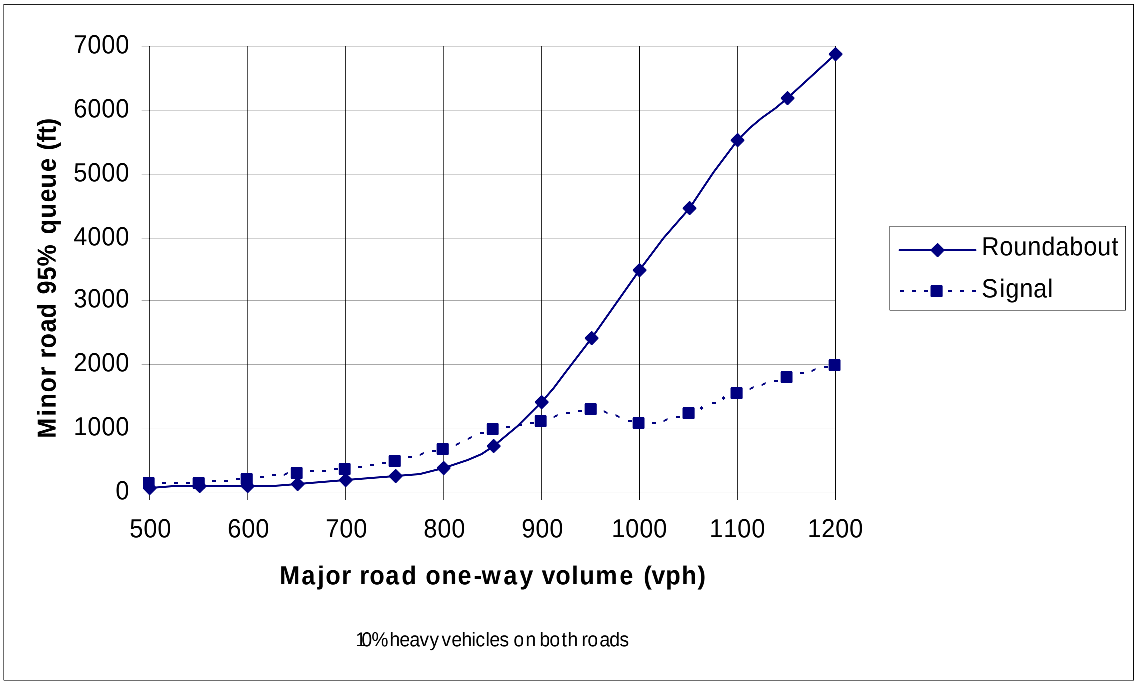


FIGURE 6.10 Minor Road Queue, Roundabout vs. TWSC Intersection



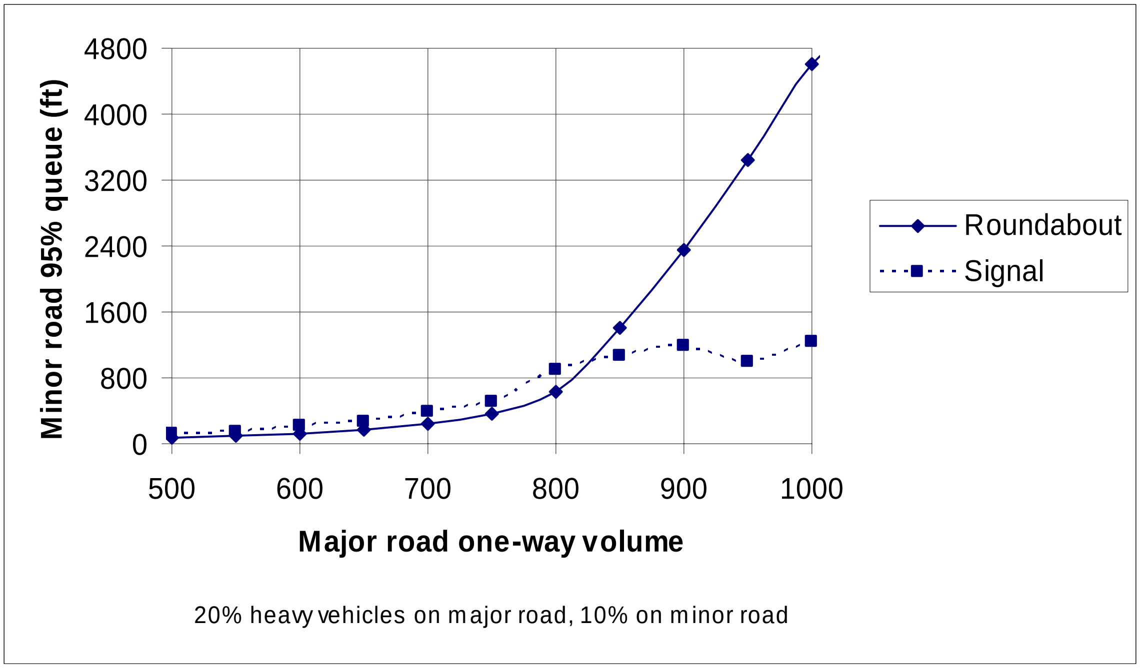


FIGURE 6.11 Minor Road Queue, Roundabout vs. Signalized Intersection

6.4 EMISSION RATES

<u>CO emissions</u>

Figure 6.12 shows CO emission rates for a roundabout versus a TWSC intersection. For 10% HV on both roads, the TWSC intersection provides less CO emissions up to an MRV of 339 vph; this threshold volume decreases to 213 vph when heavy vehicles are increased to 20% on the major road. Figure 6.13 shows CO emissions for a roundabout versus a signalized intersection. The roundabout provides less CO emissions below threshold volumes of 795 vph for 10% HV on both roads, and 725 vph for 20% HV on the major road.

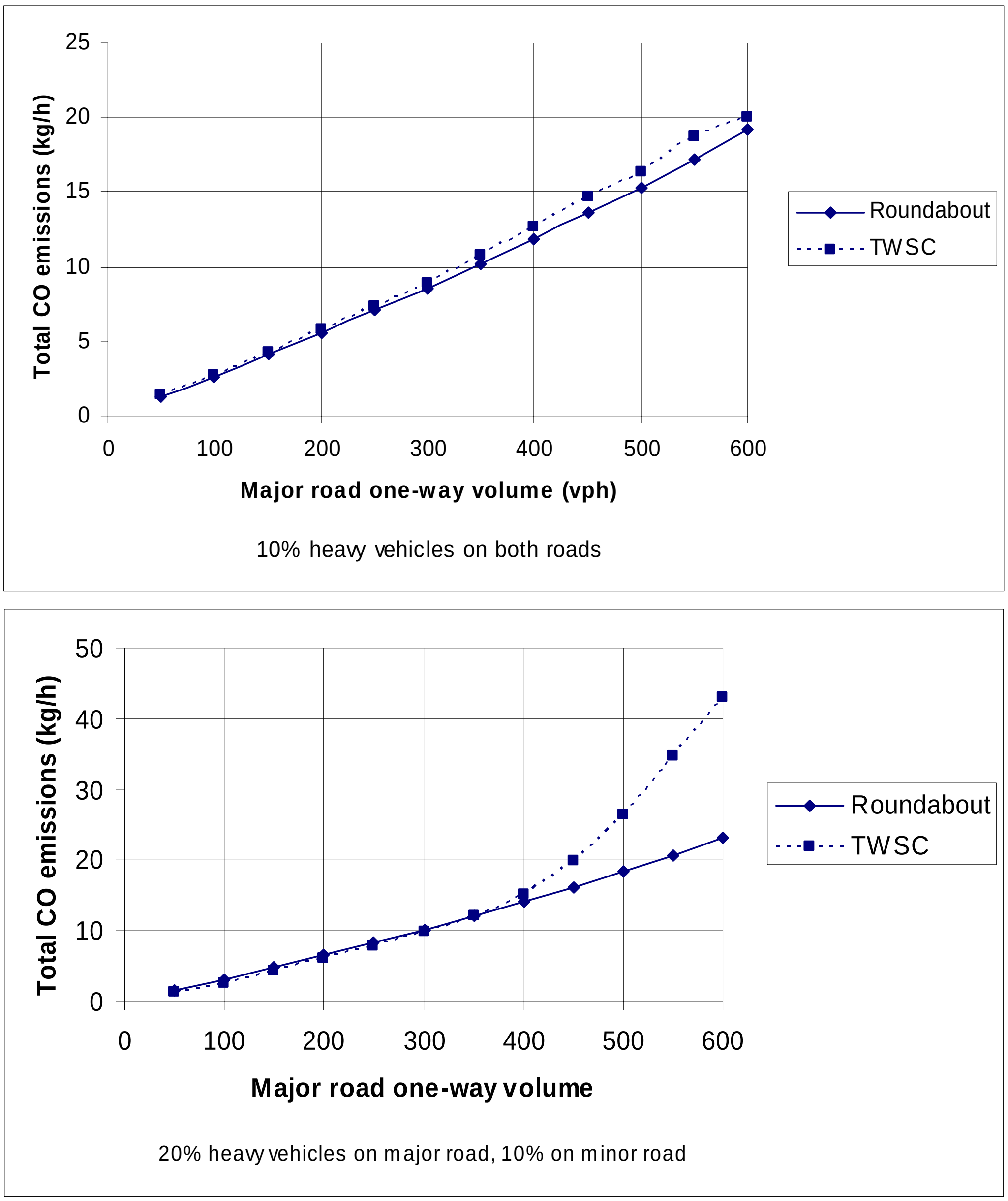
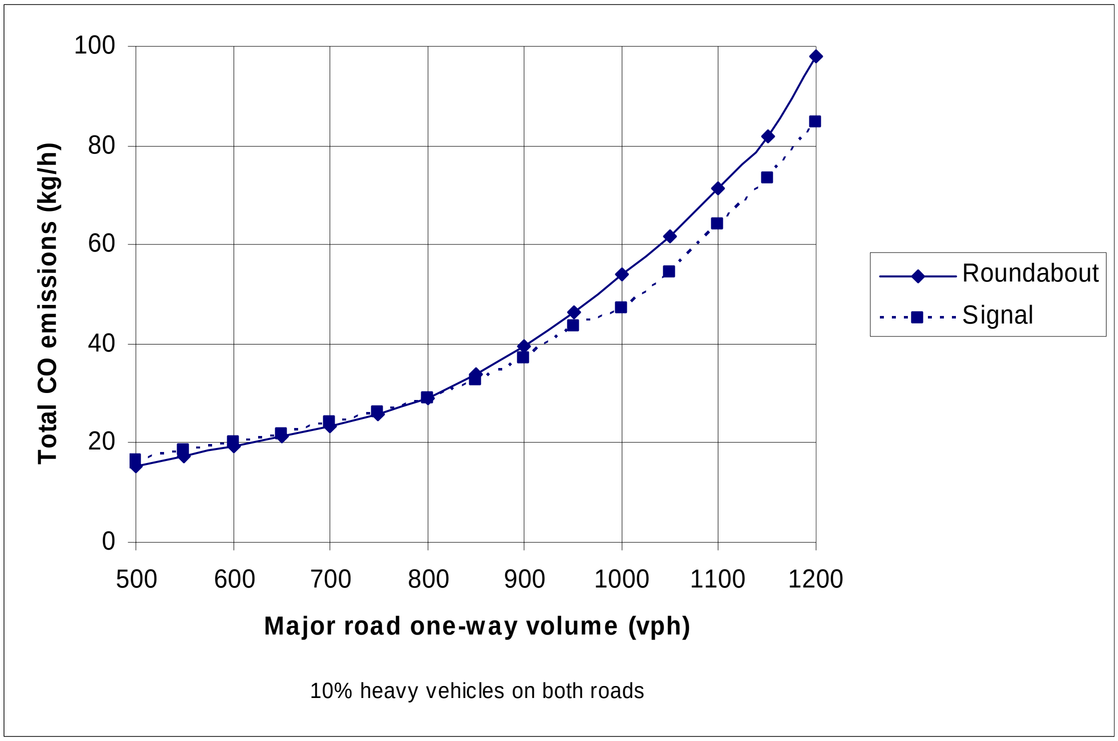


FIGURE 6.12 Total CO Emissions, Roundabout vs. TWSC Intersection



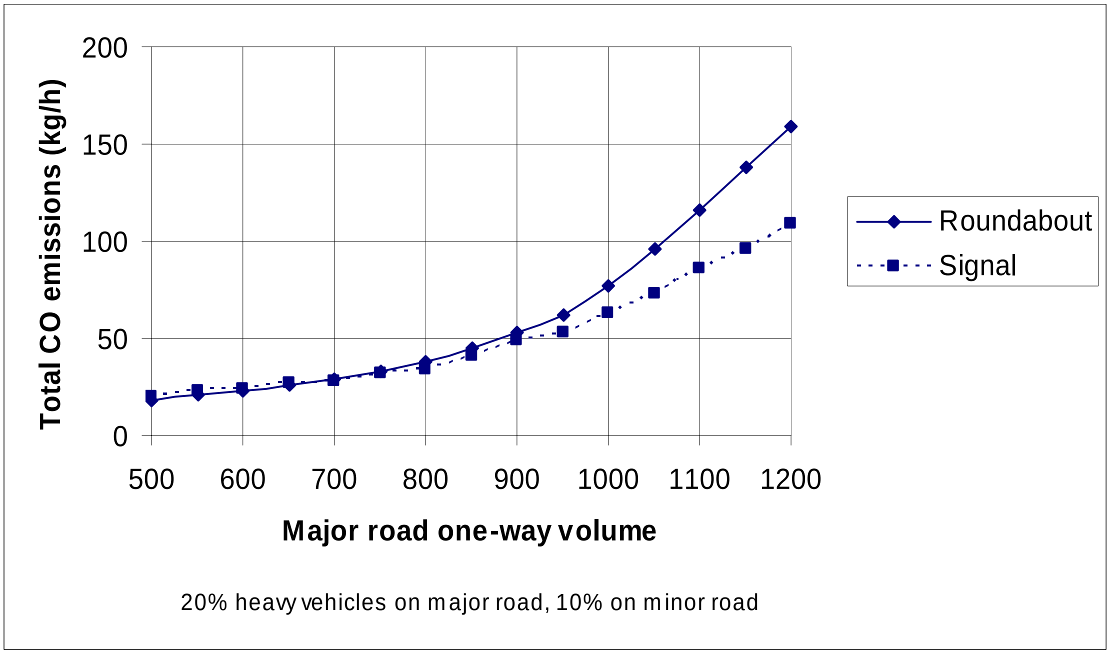


FIGURE 6.13 Total CO Emissions, Roundabout vs. Signalized Intersection

CO_2 emissions

Figure 6.14 presents this MOE comparison for a roundabout versus a TWSC intersection. The TWSC intersection has lower associated CO_2 emissions than the roundabout up until an MRV of 229 vph for 10% HV on both roads, and 336 vph when the HV is increased to 20% on the major road. Figure 6.15 shows the same comparison for a roundabout versus a signalized intersection. It can be seen that the roundabout provides less CO2 emissions below threshold volumes of 903 vph for 10% HV on both roads, and 835 vph for 20% HV on the major road.

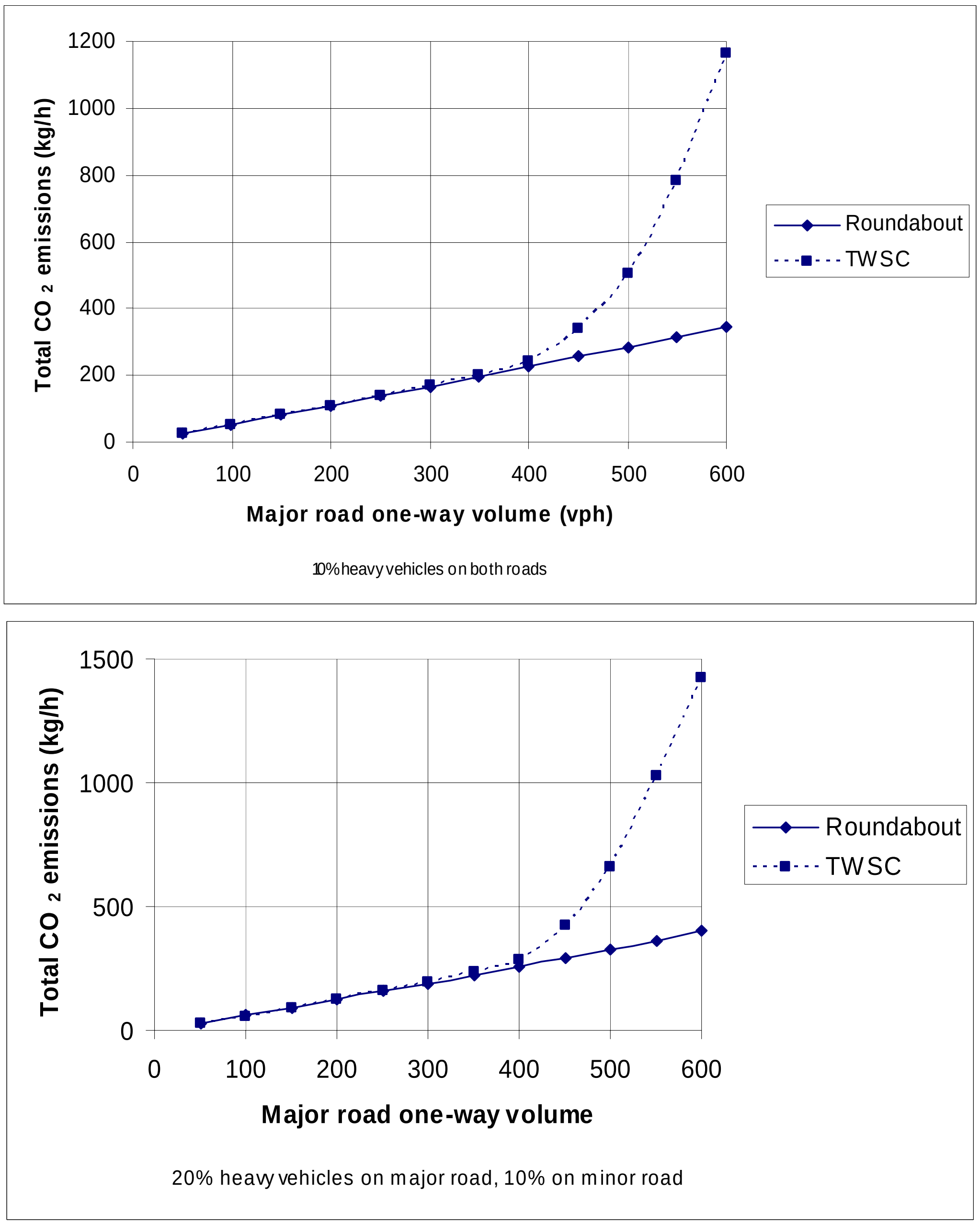


FIGURE 6.14 Total CO_2 Emissions, Roundabout vs. TWSC Intersection

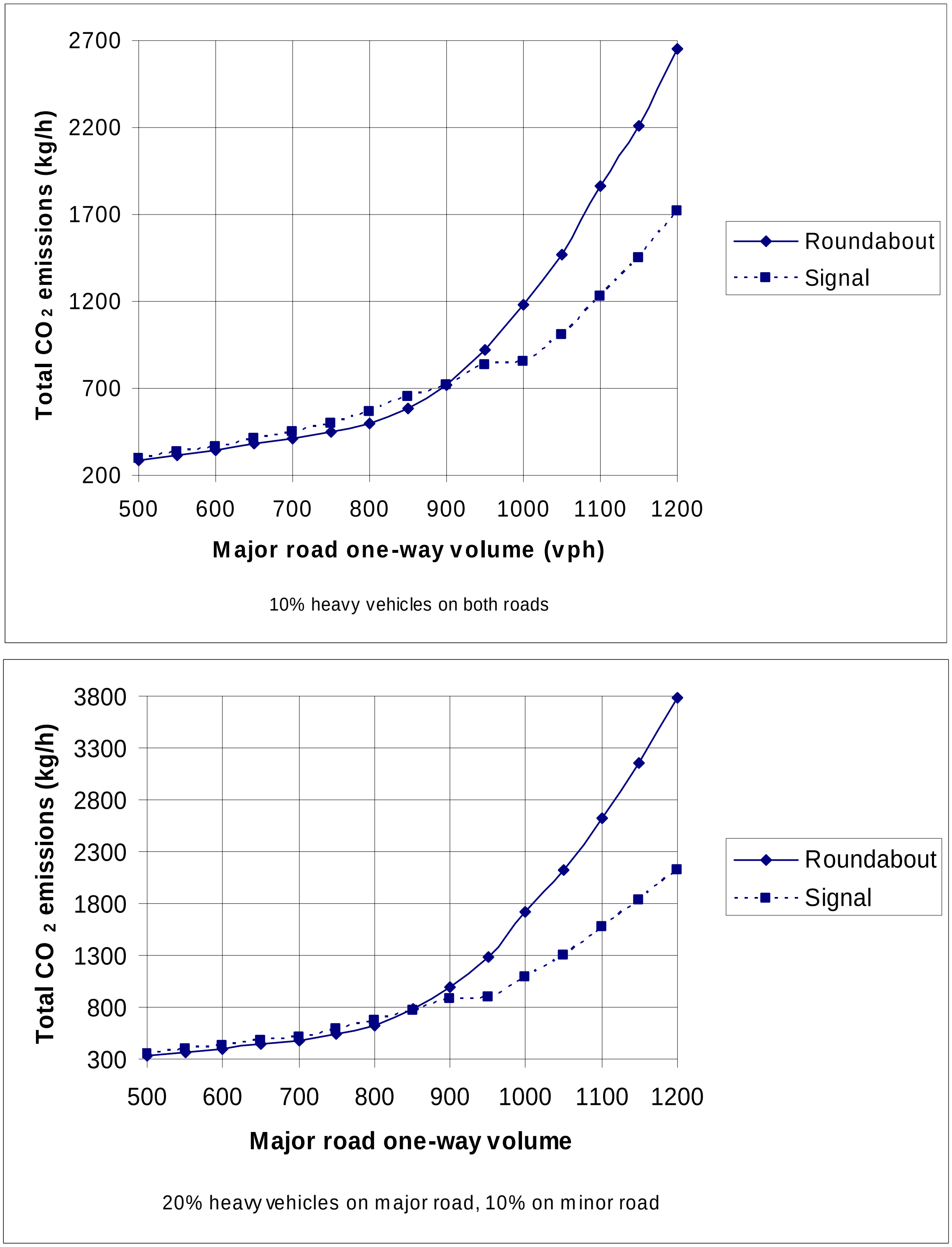


FIGURE 6.15 Total CO_2 Emissions, Roundabout vs. Signalized Intersection

<u>NO_X emissions</u>

The comparison of NO_X emissions for a roundabout versus a TWSC intersection is presented in Figure 6.16. Up until an MRV of 334 vph, the TWSC intersection provides less emissions than the roundabout for 10% HV on both roads; this threshold volume decreases to 250 vph when the HV percentage is increased to 20% on the major road. Figure 6.17 shows this MOE for a roundabout versus a signalized intersection. The threshold volumes for this comparison are 833 vph for 10% HV on both roads, and 683 vph for 20% HV on the major road and 10% HV on the minor road.

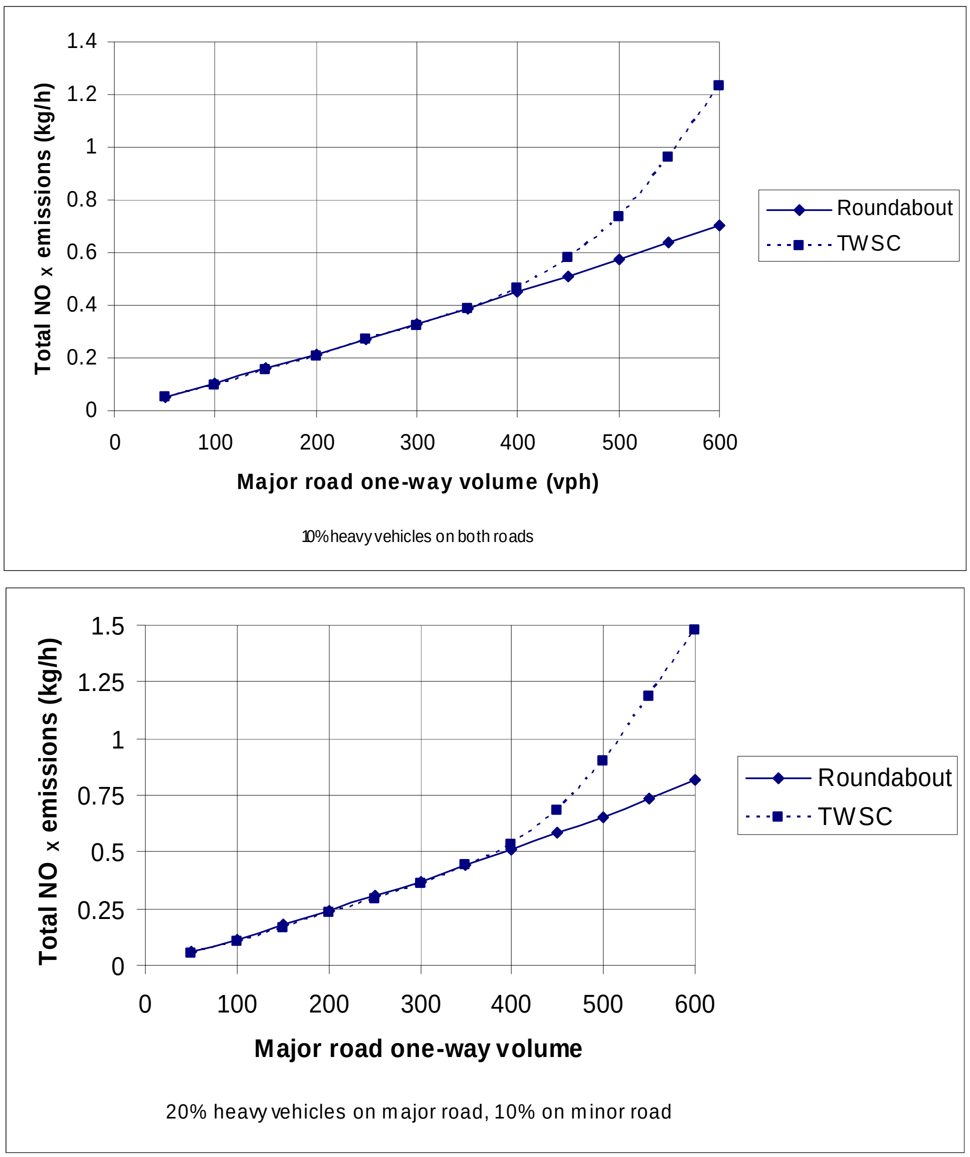
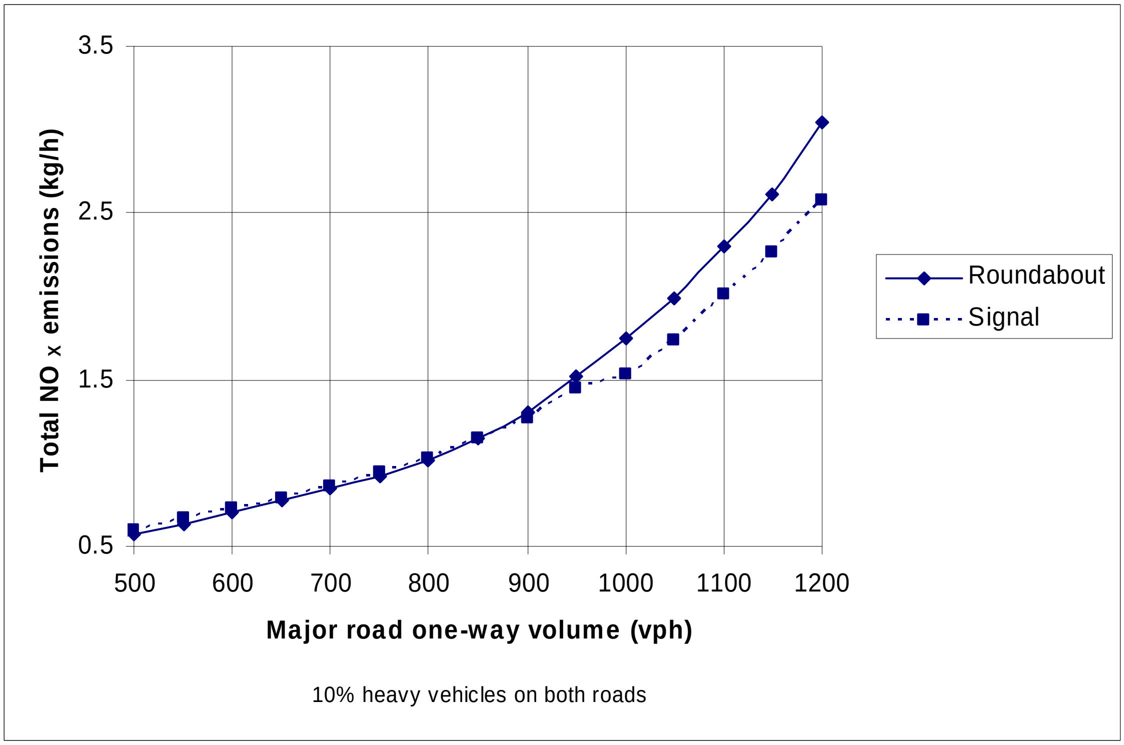


Figure 6.16 Total NO_X Emissions, Roundabout vs. TWSC Intersection



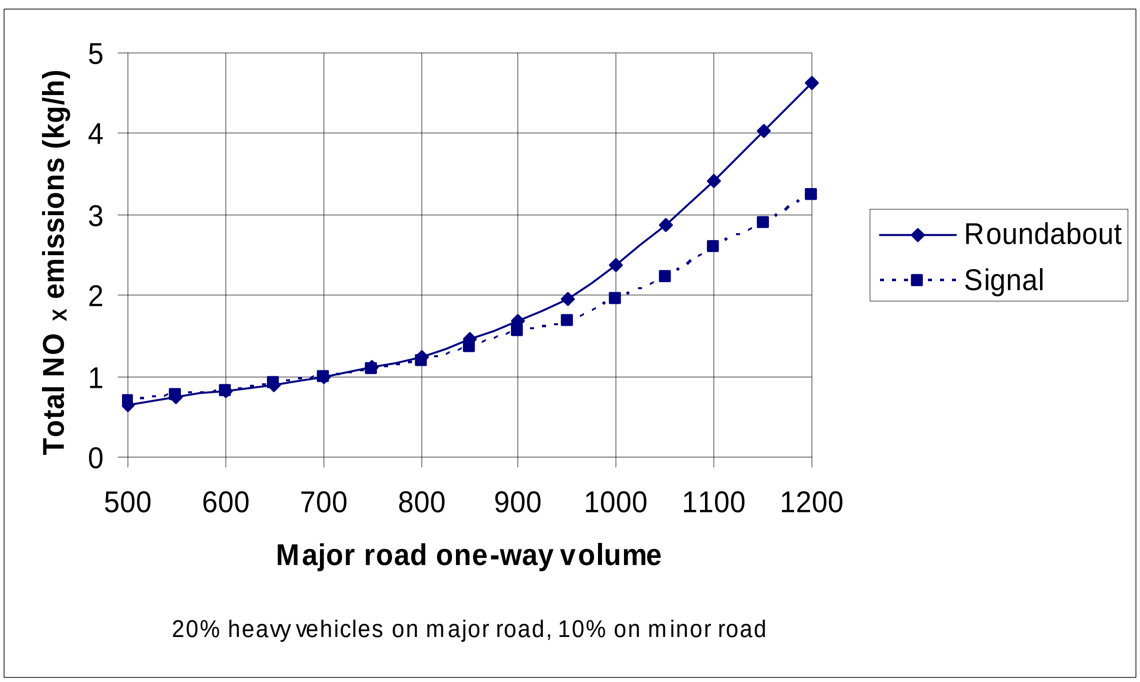


FIGURE 6.17 Total NO_X Emissions, Roundabout vs. Signalized Intersection

HC emissions

Figure 6.18 presents this comparison for a roundabout versus a TWSC intersection. The TWSC intersection provides fewer emissions than the roundabout below threshold volumes of 225 vph for 10% HV on both roads and 230 vph for 20% HV on the major road. Figure 6.19 presents the HC emissions comparison of a roundabout versus a signalized intersection. The roundabout provides fewer emissions than the signal in both heavy vehicle situations; the threshold volumes are 905 vph for 10% HV on both roads, and 834 vph for 20% HV on the major road.

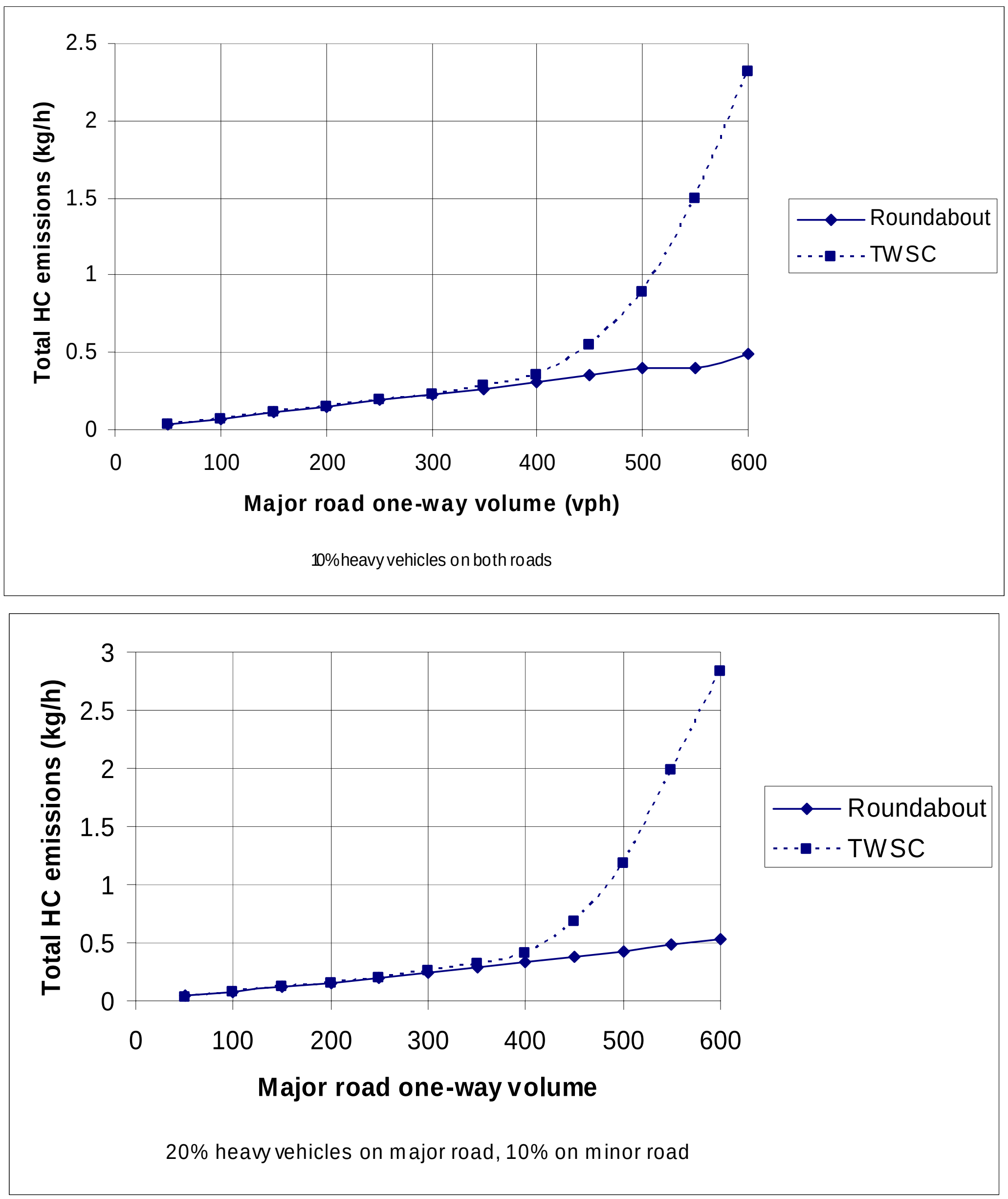
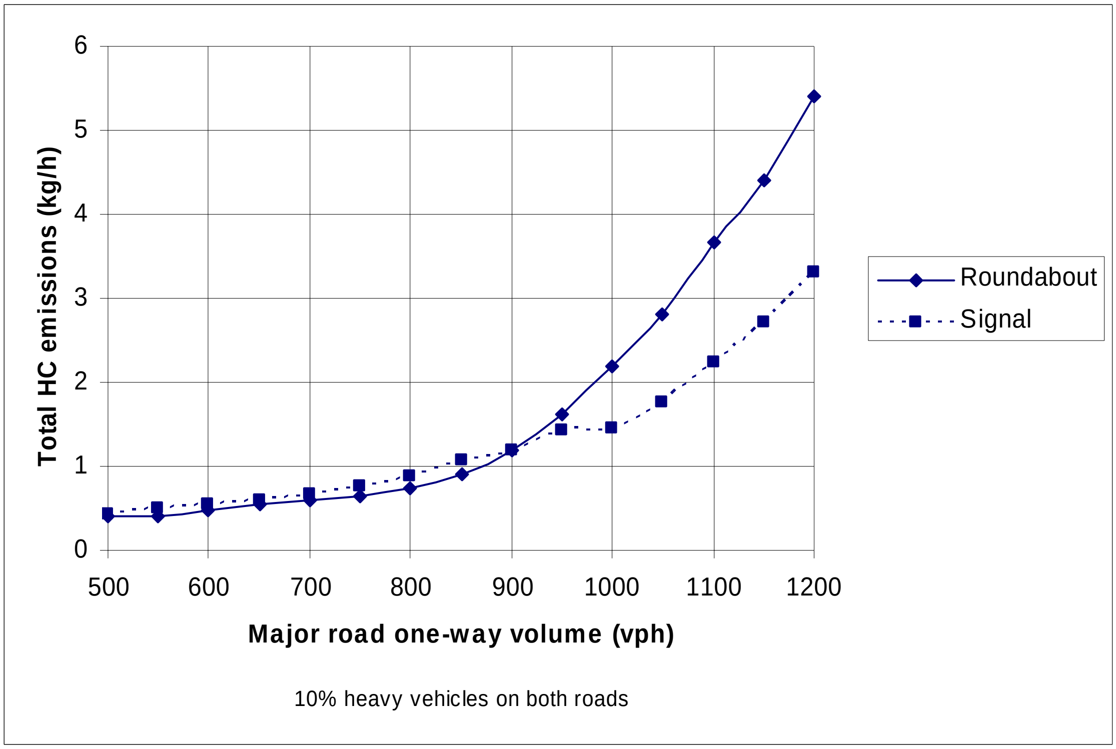


FIGURE 6.18 Total HC Emissions, Roundabout vs. TWSC Intersection



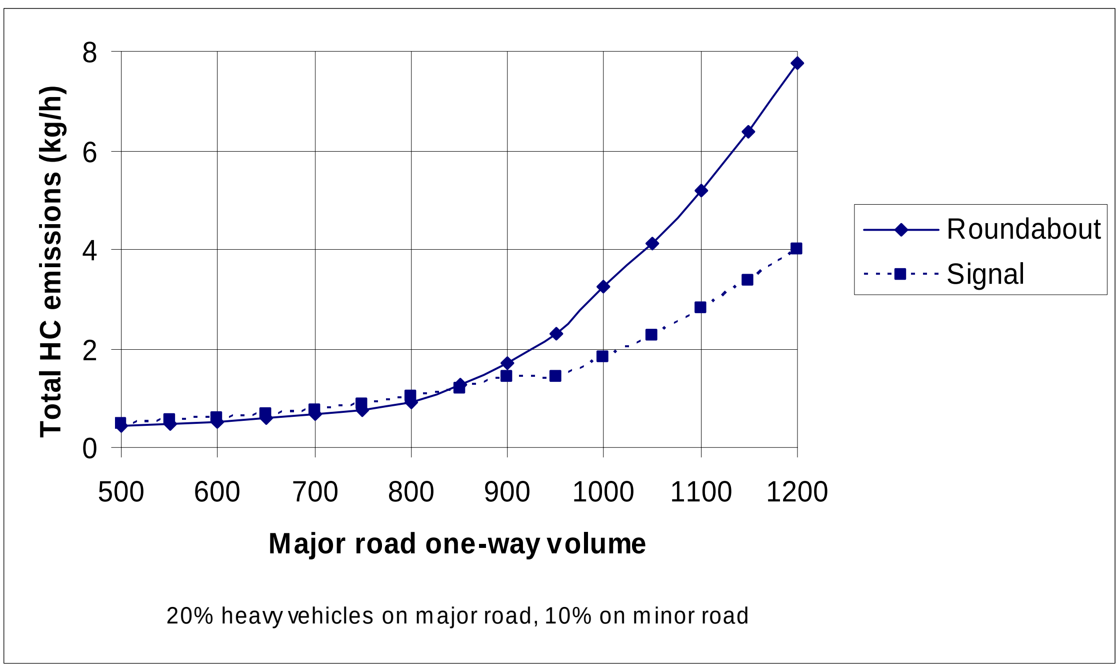


FIGURE 6.19 Total HC Emissions, Roundabout vs. Signalized Intersection

The following tables summarize the major road one-way volume thresholds determined from the SIDRA analyses for all MOEs considered. Table 6.1 shows thresholds under heavy vehicle conditions of 10% on both roads. Table 6.2 shows thresholds when heavy vehicles have been increased to 20% on the major road while remaining at 10% on the minor road.

TABLE 6.1 Threshold Volumes using Observed Maryland/Delaware Gap Acceptance Parameters for 10% Heavy Vehicles on Both Roads

	Roundabout vs. TWSC		*Roundabout vs. Signal*	
Measure of Effectiveness	*Advantage*	*Maximum MRV (vph)*	*Advantage*	*Maximum MRV (vph)*
Intersection Capacity (vph)	Roundabout	**	Roundabout	809
Major Road Capacity (vph)	N/A	N/A	Roundabout	**
Minor Road Capacity (vph)	Roundabout	**	Roundabout	805
Major Road Delay (s)	N/A	N/A	Roundabout	**
Minor Road Delay (s)	Roundabout	**	Roundabout	833
Major Road 95% Queue (ft)	N/A	N/A	Roundabout	**
Minor Road 95% Queue (ft)	Roundabout	**	Roundabout	871
Total HC Emissions (kg/h)	TWSC	225	Roundabout	905
Total CO Emissions (kg/h)	TWSC	339	Roundabout	795
Total NO_X Emissions (kg/h)	TWSC	334	Roundabout	833
Total CO_2 Emissions (kg/h)	TWSC	229	Roundabout	903

* Major One-Way Road Volume up to which the indicated intersection type shows better performance
** Intersection type shows better performance throughout the entire range of volumes considered

TABLE 6.2 Threshold Volumes using Observed Maryland/Delaware Gap Acceptance Parameters for 20% Heavy Vehicles on the Major Road, 10% on the Minor Road

	Roundabout vs. TWSC		*Roundabout vs. Signal*	
Measure of Effectiveness	*Advantage*	*Maximum MRV (vph)*	*Advantage*	*Maximum MRV (vph)*
Intersection Capacity (vph)	Roundabout	**	Roundabout	761
Major Road Capacity (vph)	N/A	N/A	Roundabout	936
Minor Road Capacity (vph)	Roundabout	**	Roundabout	766
Major Road Delay (s)	N/A	N/A	Roundabout	1111
Minor Road Delay (s)	Roundabout	**	Roundabout	802
Major Road 95% Queue (ft)	N/A	N/A	Roundabout	1114
Minor Road 95% Queue (ft)	Roundabout	**	Roundabout	817
Total HC Emissions (kg/h)	TWSC	230	Roundabout	834
Total CO Emissions (kg/h)	TWSC	213	Roundabout	725
Total NO_X Emissions (kg/h)	TWSC	250	Roundabout	683
Total CO_2 Emissions (kg/h)	TWSC	336	Roundabout	835

* Major One-Way Road Volume up to which the indicated intersection type shows better performance
** Intersection type shows better performance throughout the entire range of volumes considered

7.0 COMPARISON OF ROUNDABOUT MEASURES OF EFFECTIVENESS USING SIDRA VS. MARYLAND GAPS

The critical gap and follow-up times determined from the data collection are smaller than those recommended by the HCM and presented in Section 2.2.1 of this report. The default values used in the HCM version of the SIDRA software are based upon calibrations of SIDRA model parameters using the HCM defaults. While the default gap acceptance parameters are determined as functions of roundabout geometry and flow conditions and so vary throughout the analysis, they remained greater than the observed critical gap and follow-up values of 3.9 and 2.0 s, respectively.

The following sections compare the measures of effectiveness provided for the roundabouts with the observed Maryland/Delaware and the SIDRA default gap acceptance parameters for both heavy vehicle scenarios. In general, and as would be expected, the reduced critical gap and follow-up times allowed the roundabout to provide a greater capacity and reduced the delay time and queue length experienced by entering vehicles. The column heading *% change* in Table 7.1 denotes the increase or decrease occurring when the SIDRA defaults have been replaced by the observed gap acceptance values.

7.1 10% Heavy Vehicles on Both Roads

The effective intersection capacity and capacities of both the major and minor roads are presented in Table 7.1. It can be seen that all three capacities are increased by a relatively constant amount of approximately 23% when the SIDRA default values are replaced by those obtained for Maryland/Delaware. This is true up to an MRV of approximately 800 vph, where the percent difference then begins to decrease. This can also be observed in Figures 7.1 through 7.3, though as Figure 7.2 shows, the major road capacity does not experience a crossover point as the effective intersection and minor road capacities do.

TABLE 7.1 Effective Intersection, Major Road, and Minor Road Capacities

MRV	*Effective intersection capacity (vph)*			*Major road capacity (vph)*			*Minor road capacity (vph)*		
(vph)	*Md/De*	*SIDRA*	*% change*	*Md/De*	*SIDRA*	*% change*	*Md/De*	*SIDRA*	*% change*
50	5023	4127	21.7	1654	1359	21.7	1566	1246	25.7
100	4880	3985	22.5	1637	1337	22.4	1557	1235	26.1
150	4744	3875	22.4	1578	1288	22.5	1457	1155	26.1
200	4637	3777	22.8	1548	1261	22.8	1409	1119	25.9
250	4517	3681	22.7	1502	1224	22.7	1320	1052	25.5
300	4392	3577	22.8	1467	1195	22.8	1261	1012	24.6
350	4269	3480	22.7	1423	1160	22.7	1178	952	23.7
400	4155	3390	22.6	1384	1129	22.6	1112	906	22.7
450	4026	3293	22.3	1341	1097	22.2	1037	854	21.4
500	3907	3202	22.0	1303	1068	22.0	967	804	20.3
550	3778	3106	21.6	1260	1036	21.6	892	749	19.1
600	3660	3018	21.3	1220	1006	21.3	820	695	18.0
650	3531	2925	20.7	1176	974	20.7	740	632	17.1
700	3402	2832	20.1	1135	736	54.2	668	367	82.0
750	3292	2755	19.5	1097	918	19.5	591	525	12.6
800	3134	2681	16.9	1063	841	26.4	522	420	24.3
850	2804	2609	7.5	1025	869	18.0	468	462	1.3
900	2566	2563	0.1	1042	946	10.1	427	472	-9.5
950	2328	2577	-9.7	1081	859	25.8	388	468	-17.1
1000	2109	2588	-18.5	1117	862	29.6	352	461	-23.6
1050	1928	2578	-25.2	1147	860	33.4	321	458	-29.9
1100	1733	2584	-32.9	1177	861	36.7	289	453	-36.2
1150	1657	2576	-35.7	1183	859	37.7	276	451	-38.8
1200	1600	2584	-38.1	1171	861	36.0	267	444	-39.9

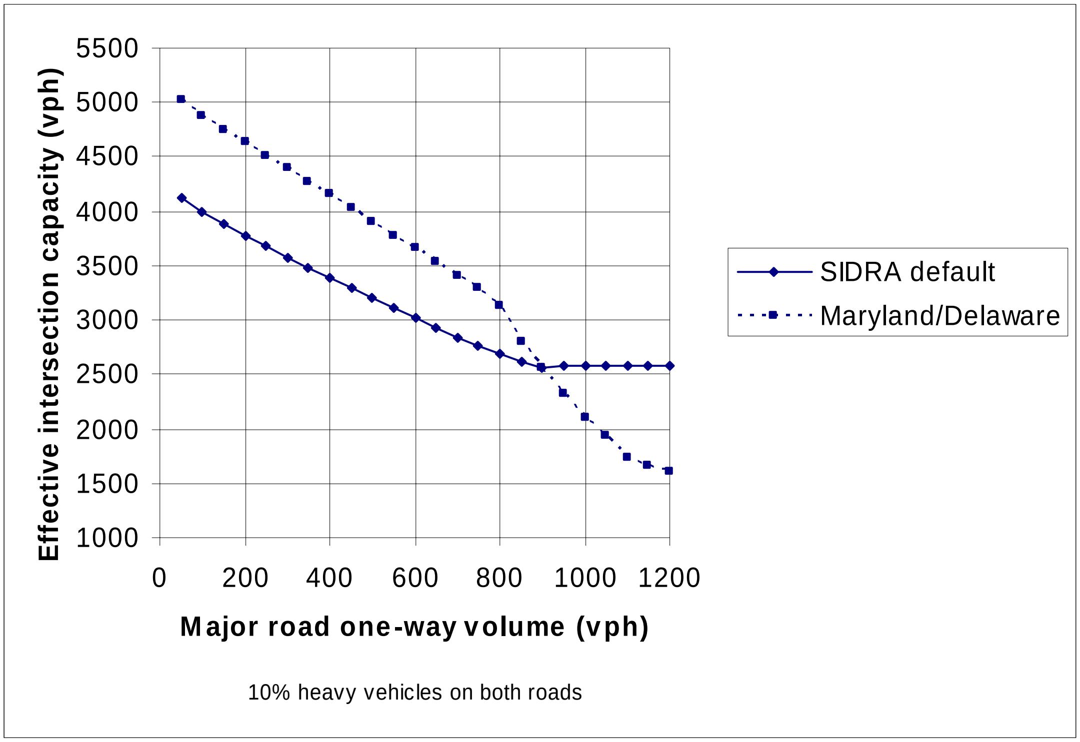


FIGURE 7.1 Roundabout Effective Intersection Capacity

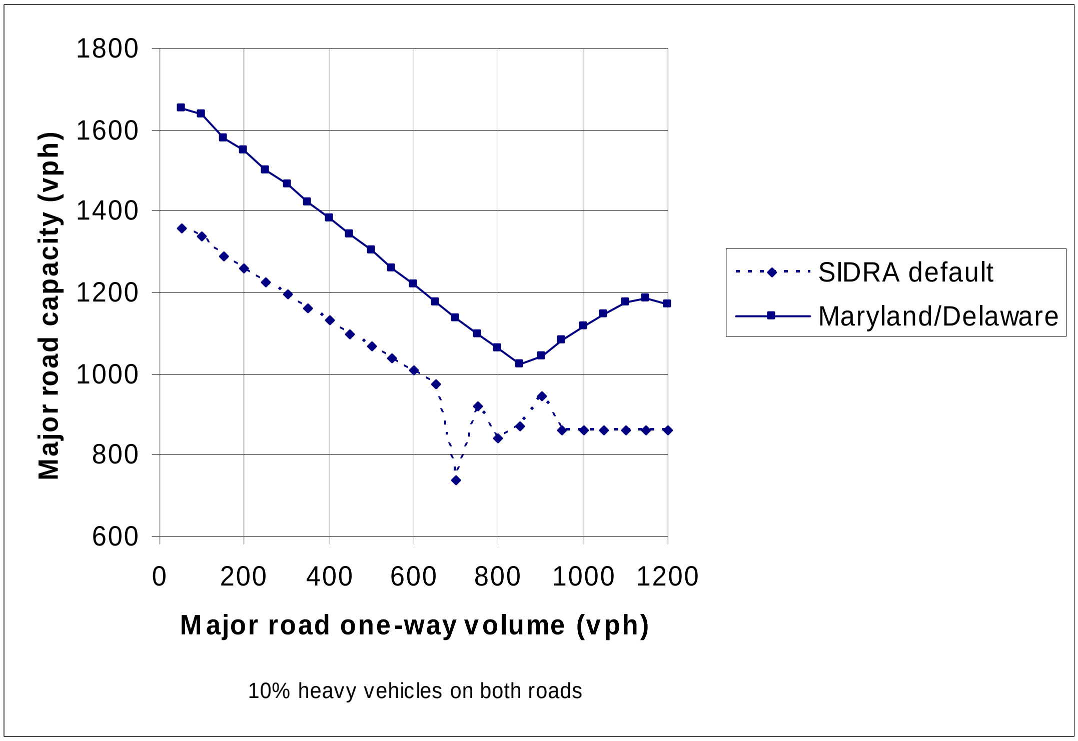


FIGURE 7.2 Roundabout Major Road Capacity

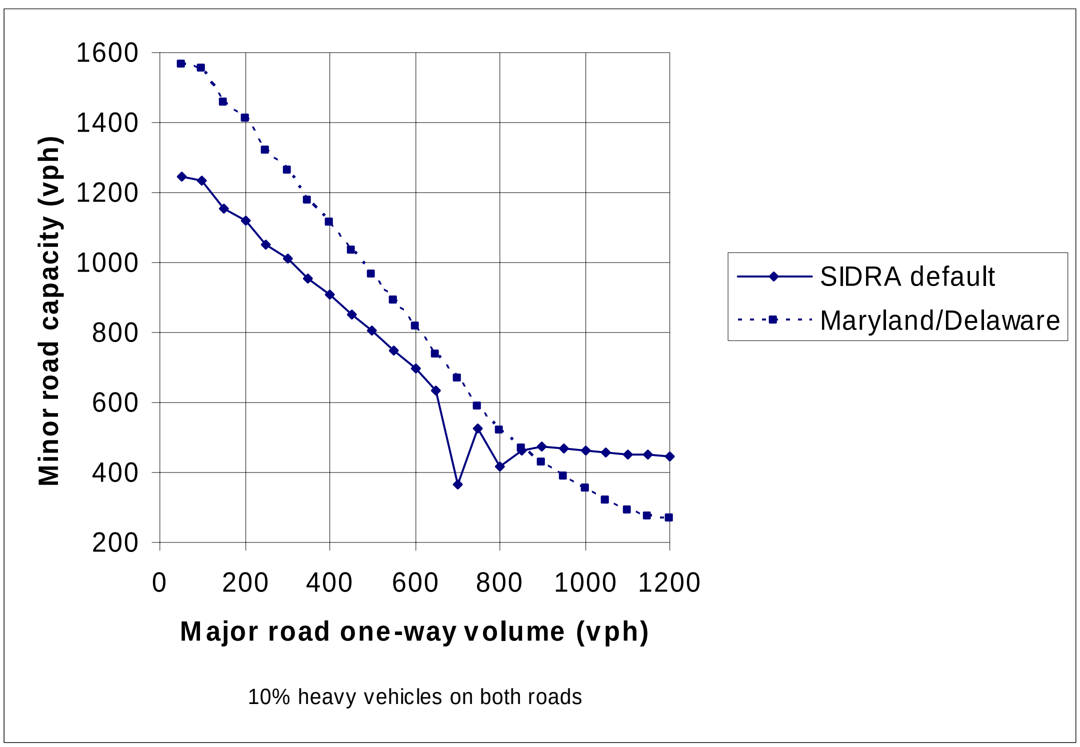


FIGURE 7.3 Roundabout Minor Road Capacity

The major and minor road average delays are presented in Table 7.2. A decrease in both delays was observed when the SIDRA default gap parameters were replaced with those observed for Maryland/Delaware. This decrease increased with increasing major road one-way volume.

TABLE 7.2 Major and Minor Road Delay

MRV	*Major road average delay (s)*			*Minor road average delay (s)*		
(vph)	*Md/De*	*SIDRA*	*% change*	*Md/De*	*SIDRA*	*% change*
50	6.0	6.1	-1.6	7.0	7.1	-1.4
100	6.1	6.2	-1.6	7.1	7.3	-2.7
150	6.2	6.4	-3.1	7.4	7.7	-3.9
200	6.3	6.6	-4.5	7.6	8.0	-5.0
250	6.5	6.8	-4.4	7.9	8.4	-6.0
300	6.6	7.0	-5.7	8.2	8.7	-5.7
350	6.8	7.2	-5.6	8.6	9.2	-6.5
400	7.0	7.5	-6.7	9.0	9.6	-6.3
450	7.2	7.8	-7.7	9.4	10.1	-6.9
500	7.4	8.1	-8.6	10.0	10.7	-6.5
550	7.7	8.7	-11.5	10.6	11.3	-6.2
600	8.0	10.0	-20.0	11.3	12.6	-10.3
650	8.7	11.8	-26.3	12.9	15.2	-15.1
700	9.9	14.4	-31.3	15.7	18.9	-16.9
750	11.7	19.4	-39.7	21.3	26.1	-18.4
800	14.4	31.1	-53.7	33.6	41.1	-18.2
850	20.0	69.8	-71.3	73.5	73.5	0.0
900	23.1	124.7	-81.5	161.5	83.1	94.3
950	24.3	171.8	-85.9	309.0	124.6	148.0
1000	26.4	222.1	-88.1	493.6	181.1	172.6
1050	29.7	276.1	-89.2	691.4	227.2	204.3
1100	35.5	330.9	-89.3	949.2	292.9	224.1
1150	54.8	386.9	-85.8	1114.0	345.6	222.3
1200	95.6	438.4	-78.2	1281.6	418.6	206.2

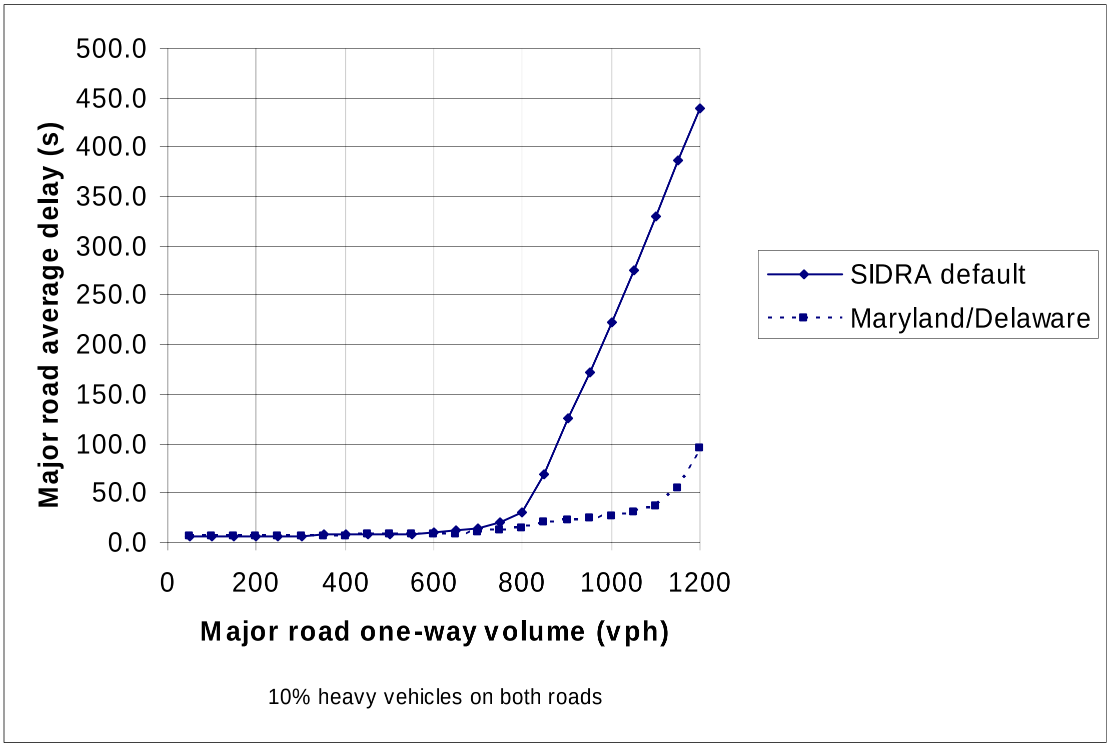


FIGURE 7.4 Roundabout Major Road Average Delay

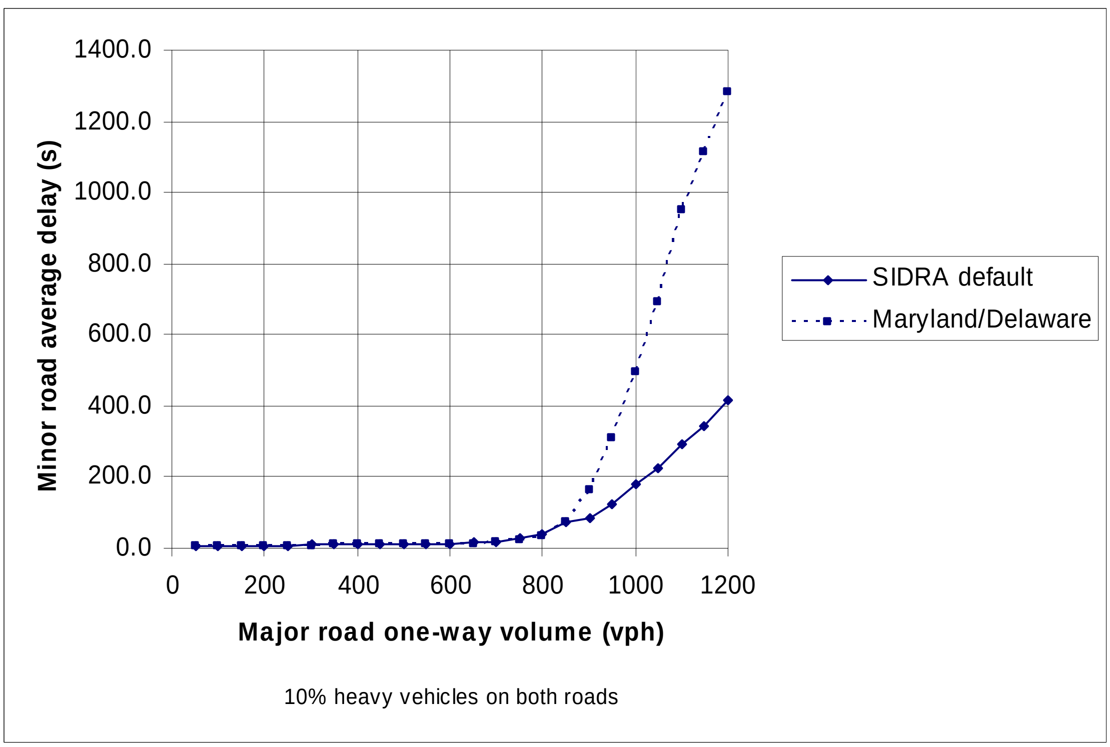


FIGURE 7.5 Roundabout Minor Road Average Delay

The major and minor road queue lengths presented in Table 7.3 follow a somewhat similar pattern where the difference change increases with increasing MRV. Both the minor road delay and queue length experience a crossover point at an MRV of 850-900 where the MOE begins to decrease, as can be seen in Figures 7.5 and 7.7.

TABLE 7.3 Major and Minor Road Queue Length

MRV	*Major road 95% queue (ft)*			*Minor road 95% queue (ft)*		
(vph)	*Md/De*	*SIDRA*	*% change*	*Md/De*	*SIDRA*	*% change*
50	5	7	-28.6	3	4	-25.0
100	11	14	-21.4	6	7	-14.3
150	18	24	-25.0	10	13	-23.1
200	26	34	-23.5	14	18	-22.2
250	35	46	-23.9	20	25	-20.0
300	45	58	-22.4	25	31	-19.4
350	57	74	-23.0	33	40	-17.5
400	71	91	-22.0	42	50	-16.0
450	87	111	-21.6	53	63	-15.9
500	104	132	-21.2	66	76	-13.2
550	125	162	-22.8	82	94	-12.8
600	149	215	-30.7	102	123	-17.1
650	186	286	-35.0	136	168	-19.0
700	241	375	-35.7	183	223	-17.9
750	313	522	-40.0	261	310	-15.8
800	409	802	-49.0	390	453	-13.9
850	584	1474	-60.4	730	736	-0.8
900	698	2316	-69.9	1408	849	65.8
950	776	3087	-74.9	2420	1225	97.6
1000	879	3917	-77.6	3498	1719	103.5
1050	1010	4784	-78.9	4444	2117	109.9
1100	1212	5695	-78.7	5518	2671	106.6
1150	1673	6597	-74.6	6181	3124	97.9
1200	2521	7464	-66.2	6869	3716	84.8

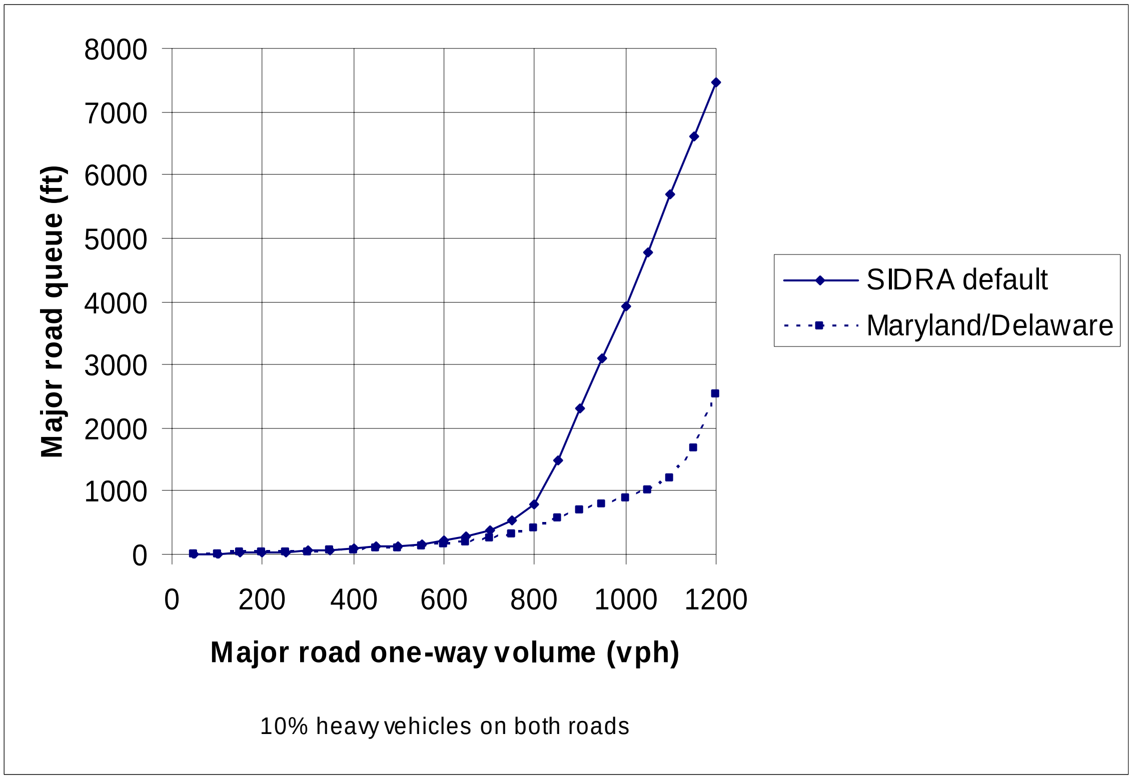


FIGURE 7.6 Roundabout Major Road Queue Length

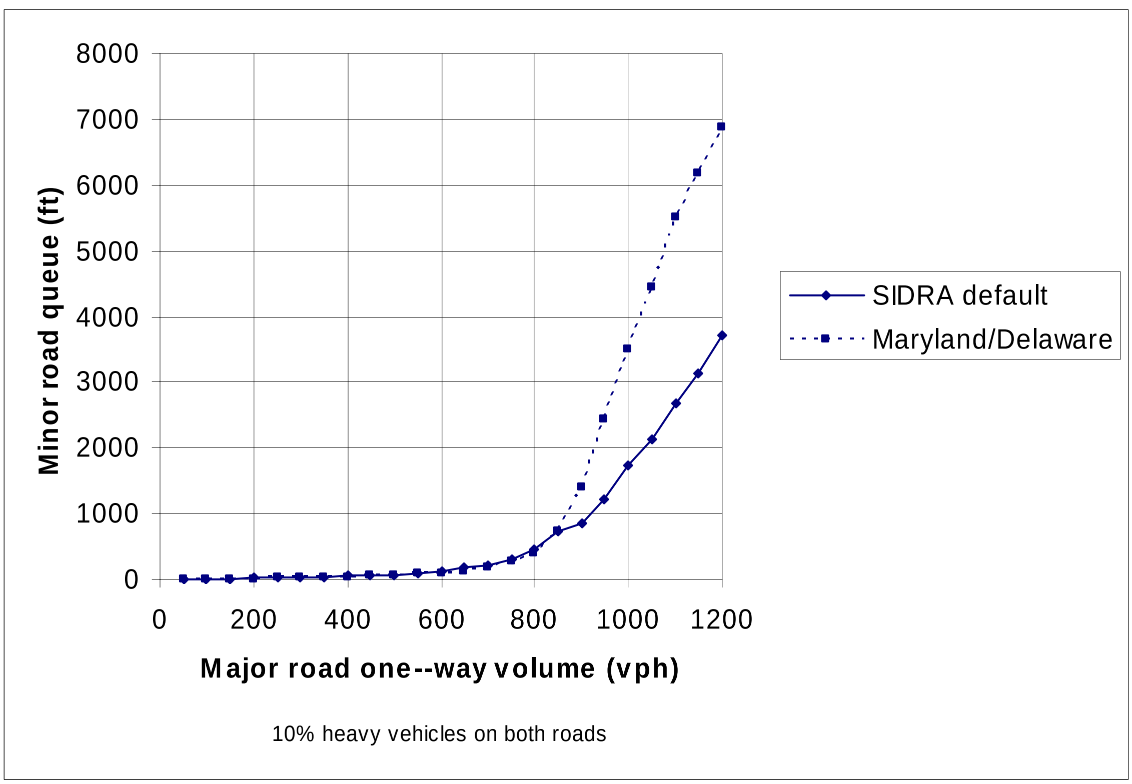


Figure 7.7 Roundabout Minor Road Queue Length

The results for the four emissions measures of effectiveness are presented in Table 7.4. It can be seen that in almost all cases the emissions were reduced with the smaller gap acceptance parameters. This corresponds to the reduced delay and therefore reduced idling times which would cause vehicle to emit more emissions.

TABLE 7.4 Roundabout Total Emissions

MRV	*Total HC emissions (kg/h)*			*Total CO emissions (kg/h)*			*Total CO_2 emissions (kg/h)*			*Total NO_X emissions (kg/h)*		
(vph)	*Md/De*	*SIDRA*	*% change*	*Md/De*	*SIDRA*	*% change*	*Md/De*	*SIDRA*	*% change*	*Md/De*	*SIDRA*	*% change*
50	0.037	0.037	0.0	1.35	1.36	-0.7	27.9	28.0	-0.4	0.054	0.054	0.0
100	0.071	0.072	-1.4	2.60	2.63	-1.1	53.1	53.3	-0.4	0.103	0.104	-1.0
150	0.111	0.112	-0.9	4.10	4.16	-1.4	82.5	82.9	-0.5	0.161	0.162	-0.6
200	0.149	0.150	-0.7	5.52	5.61	-1.6	109.8	110.4	-0.5	0.215	0.217	-0.9
250	0.189	0.191	-1.0	7.06	7.18	-1.7	138.8	139.6	-0.6	0.273	0.276	-1.1
300	0.226	0.228	-0.9	8.50	8.65	-1.7	165.3	166.3	-0.6	0.327	0.330	-0.9
350	0.268	0.271	-1.1	10.17	10.36	-1.8	195.4	196.6	-0.6	0.388	0.392	-1.0
400	0.310	0.313	-1.0	11.85	12.08	-1.9	225.1	226.6	-0.7	0.449	0.454	-1.1
450	0.353	0.358	-1.4	13.61	13.89	-2.0	255.6	257.4	-0.7	0.512	0.519	-1.3
500	0.394	0.400	-1.5	15.30	15.62	-2.0	283.9	286.1	-0.8	0.572	0.579	-1.2
550	0.400	0.447	-10.5	17.20	17.59	-2.2	315.4	318.1	-0.8	0.638	0.647	-1.4
600	0.487	0.495	-1.6	19.17	19.64	-2.4	347.1	350.5	-1.0	0.706	0.717	-1.5
650	0.537	0.551	-2.5	21.28	22.00	-3.3	380.4	386.5	-1.6	0.778	0.794	-2.0
700	0.588	0.617	-4.7	23.45	24.76	-5.3	413.0	425.1	-2.8	0.848	0.876	-3.2
750	0.649	0.703	-7.7	25.80	27.77	-7.1	451.3	474.4	-4.9	0.925	0.966	-4.2
800	0.737	0.841	-12.4	28.94	31.82	-9.1	500.3	544.8	-8.2	1.017	1.079	-5.7
850	0.913	1.170	-22.0	33.73	40.20	-16.1	588.6	699.6	-15.9	1.150	1.295	-11.2
900	1.188	1.547	-23.2	39.33	49.68	-20.8	719.6	873.5	-17.6	1.307	1.528	-14.5
950	1.621	2.001	-19.0	46.44	60.16	-22.8	922.6	1083.1	-14.8	1.515	1.793	-15.5
1000	2.180	2.546	-14.4	54.16	71.92	-24.7	1181.4	1332.9	-11.4	1.752	2.093	-16.3
1050	2.799	3.106	-9.9	61.61	83.39	-26.1	1466.6	1587.8	-7.6	1.990	2.387	-16.6
1100	3.661	3.781	-3.2	71.17	96.59	-26.3	1862.1	1895.5	-1.8	2.302	2.730	-15.7
1150	4.414	4.455	-0.9	81.93	109.19	-25.0	2205.7	2202.1	0.2	2.615	3.061	-14.6
1200	5.403	5.208	3.7	97.87	122.61	-20.2	2653.0	2544.5	4.3	3.049	3.418	-10.8

7.2 20% Heavy Vehicles on Major Road, 10% Heavy Vehicles on Minor Road

Table 7.5 presents the change in effective intersection capacity and major and minor road capacities when the heavy vehicle percentage is increased to 20% on the major road. Similar to the values in Table 7.1, the capacities are all increased by a relatively constant 23% up to an MRV of 750 vph. After which the differences in effective intersection and minor road capacities begin to decrease. Figures 7.8 through 7.10 also show these values.

TABLE 7.5 Effective Intersection, Major Road, and Minor Road Capacities

MRV	*Effective intersection capacity (vph)*			*Major road capacity (vph)*			*Minor road capacity (vph)*		
(vph)	*Md/De*	*SIDRA*	*% change*	*Md/De*	*SIDRA*	*% change*	*Md/De*	*SIDRA*	*% change*
50	4717	3877	21.7	1533	1260	21.7	1559	1240	25.7
100	4406	3610	22.0	1478	1211	22.0	1543	1224	26.1
150	4350	3555	22.4	1444	1180	22.4	1438	1139	26.3
200	4190	3421	22.5	1399	1142	22.5	1383	1100	25.7
250	4098	3341	22.7	1359	1108	22.7	1289	1030	25.1
300	3938	3209	22.7	1317	1073	22.7	1223	985	24.2
350	3837	3129	22.6	1278	1042	22.6	1135	922	23.1
400	3726	3044	22.4	1240	1013	22.4	1062	871	21.9
450	3605	2949	22.2	1199	981	22.2	981	813	20.7
500	3461	2842	21.8	1156	949	21.8	903	752	20.1
550	3344	2751	21.6	1116	918	21.6	821	695	18.1
600	3229	2666	21.1	1077	889	21.1	741	633	17.1
650	3116	2584	20.6	1037	860	20.6	655	563	16.3
700	2998	2500	19.9	1001	835	19.9	577	517	11.6
750	2902	2431	19.4	967	810	19.4	502	469	7.0
800	2698	2368	13.9	933	790	18.1	449	468	-4.1
850	2414	2321	4.0	949	773	22.8	403	478	-15.7
900	2190	2262	-3.2	980	755	29.8	364	491	-25.9
950	1962	2208	-11.1	1016	736	38.0	327	488	-33.0
1000	1730	2199	-21.3	1015	732	38.7	289	474	-39.0
1050	1694	2171	-22.0	988	724	36.5	282	475	-40.6
1100	1612	2191	-26.4	977	730	33.8	269	466	-42.3
1150	1573	2171	-27.5	948	724	30.9	262	466	-43.8
1200	1485	2186	-32.1	934	728	28.3	248	456	-45.6

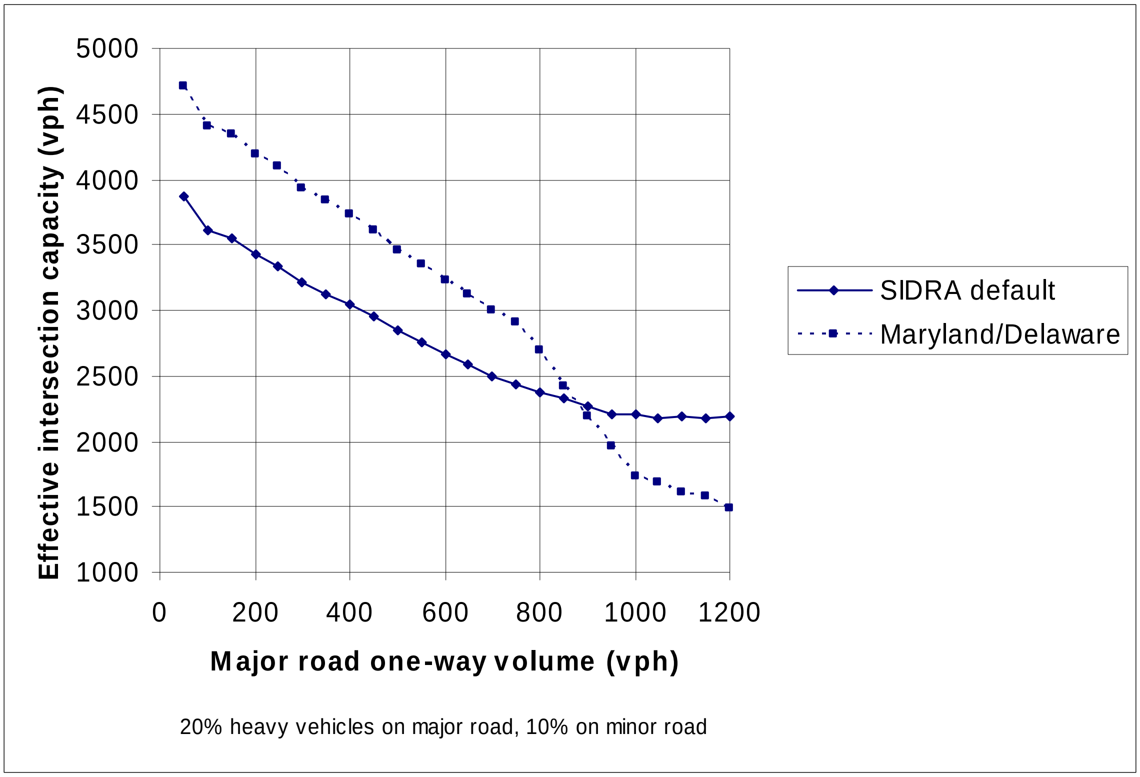


FIGURE 7.8 Roundabout Effective Intersection Capacity

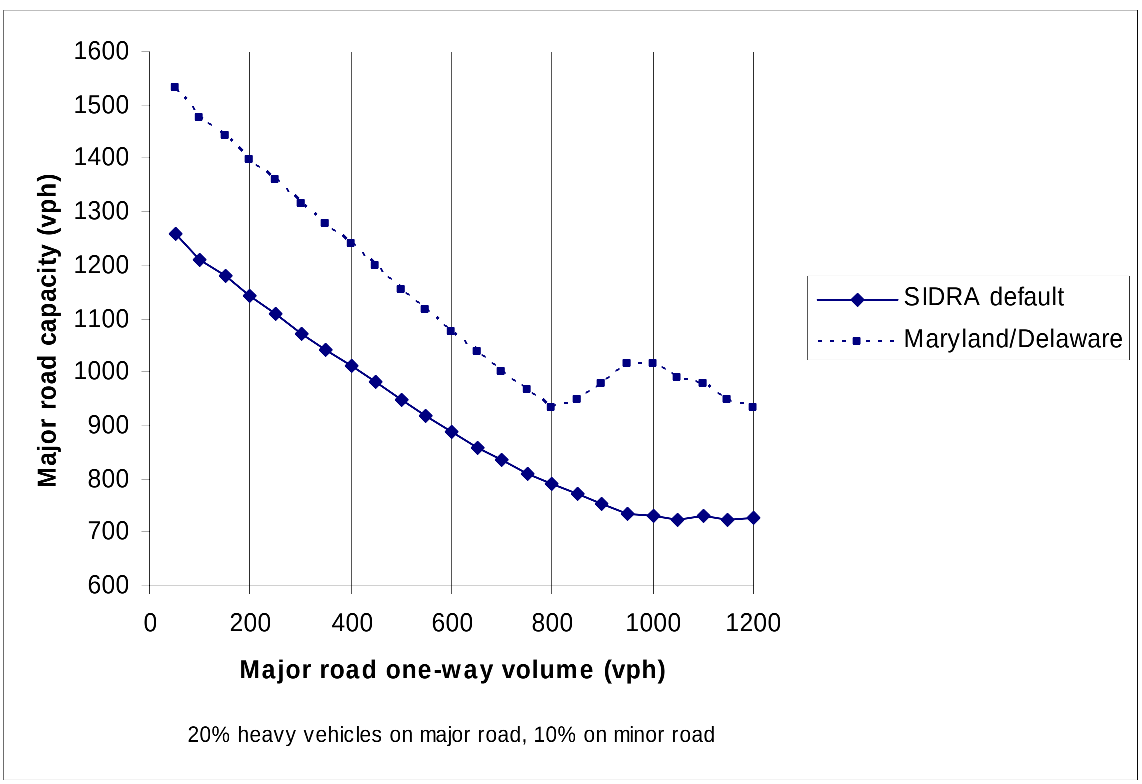


FIGURE 7.9 Roundabout Major Road Capacity

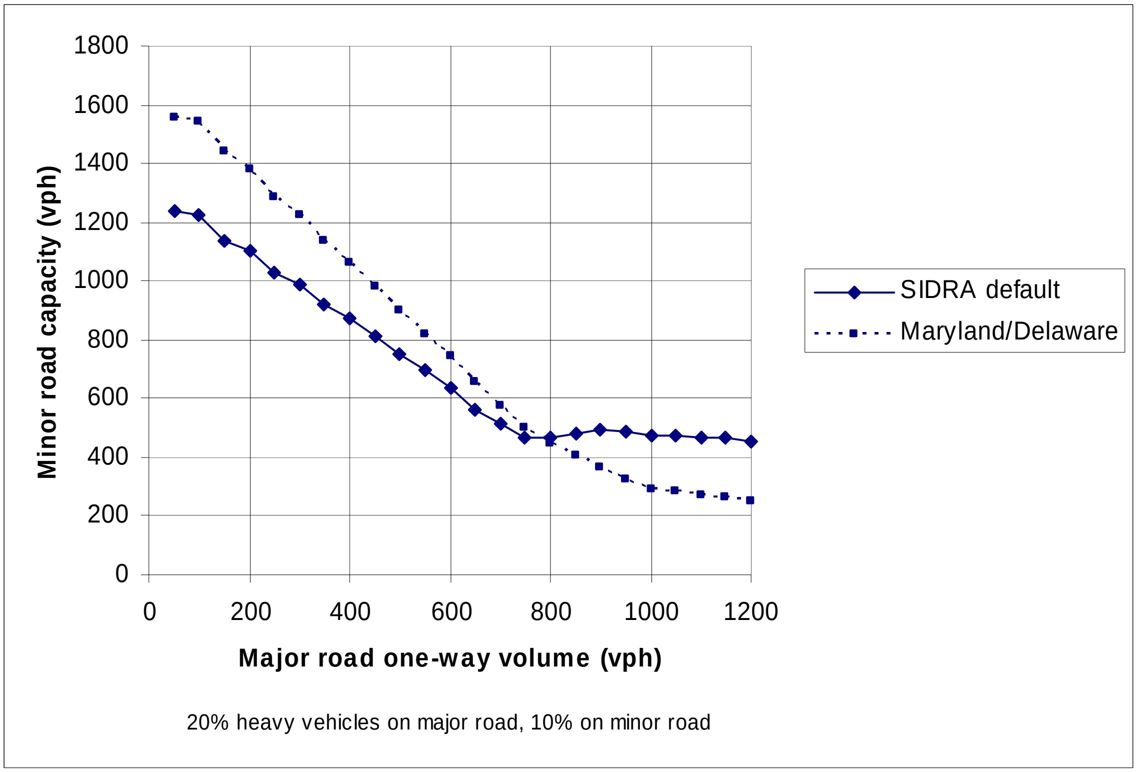


FIGURE 7.10 Roundabout Minor Road Capacity

The major and minor road average delays presented in Table 7.6 show the same pattern as in Table 7.2. The amount of decrease in delay increases with increasing MRV throughout the range of volumes analyzed, except for the minor road delay, which has a crossover point at an MRV of 800 vph.

TABLE 7.6 Major and Minor Road Delay

MRV	*Major road average delay (s)*			*Minor road average delay (s)*		
(vph)	*Md/De*	*SIDRA*	*% change*	*Md/De*	*SIDRA*	*% change*
50	6.2	6.2	0.0	7.0	7.1	-1.4
100	6.3	6.4	-1.6	7.2	7.4	-2.7
150	6.4	6.6	-3.0	7.4	7.8	-5.1
200	6.6	6.8	-2.9	7.7	8.7	-11.5
250	6.7	7.1	-5.6	8.0	8.5	-5.9
300	6.9	7.4	-6.8	8.3	8.9	-6.7
350	7.1	7.7	-7.8	8.8	9.4	-6.4
400	7.4	8.0	-7.5	9.2	9.9	-7.1
450	7.6	8.4	-9.5	9.8	10.5	-6.7
500	8	9.1	-12.1	10.4	11.1	-6.3
550	8.3	10.6	-21.7	11.2	12.0	-6.7
600	9.2	12.7	-27.6	12.2	14.2	-14.1
650	10.7	16.1	-33.5	15.3	18.2	-15.9
700	12.9	23.0	-43.9	20.2	23.9	-15.5
750	16.4	42.6	-61.5	32.8	38.8	-15.5
800	24.3	95.0	-74.4	66.9	49.3	35.7
850	30.9	169.6	-81.8	172.0	59.7	188.1
900	35.9	253.6	-85.8	319.3	65.6	386.7
950	40.2	344.3	-88.3	525.3	95.6	449.5
1000	68.1	413.4	-83.5	792.0	155.6	409.0
1050	130.1	491.8	-73.5	906.2	192.2	371.5
1100	188	547.4	-65.7	1083.9	261.5	314.5
1150	269.2	624.8	-56.9	1223.5	307.9	297.4
1200	334.3	678.7	-50.7	1443.3	385.7	274.2

The same pattern seen in Tables 7.2 and 7.6 can be observed for the major and minor road 95% queue lengths, where the decrease in queue length increases with increasing MRV. This data is presented in Table 7.7. The minor road queue length has a crossover point at the same MRV as the minor road delay (800 vph).

TABLE 7.7 Major and Minor Road Queue Length

MRV	*Major road 95% queue (ft)*			*Minor road 95% queue (ft)*		
(vph)	*Md/De*	*SIDRA*	*% change*	*Md/De*	*SIDRA*	*% change*
50	6	8	-25.0	3	4	-25.0
100	13	17	-23.5	6	7	-14.3
150	21	28	-25.0	10	13	-23.1
200	31	40	-22.5	14	18	-22.2
250	42	54	-22.2	20	25	-20.0
300	55	70	-21.4	26	32	-18.8
350	69	89	-22.5	35	42	-16.7
400	86	110	-21.8	45	53	-15.1
450	105	135	-22.2	58	67	-13.4
500	129	170	-24.1	72	83	-13.3
550	156	231	-32.5	92	106	-13.2
600	200	310	-35.5	118	146	-19.2
650	265	423	-37.4	169	206	-18.0
700	351	615	-42.9	236	272	-13.2
750	473	1030	-54.1	365	410	-11.0
800	704	1858	-62.1	635	513	23.8
850	906	2967	-69.5	1411	631	123.6
900	1074	4185	-74.3	2341	710	229.7
950	1234	5446	-77.3	3449	1002	244.2
1000	1822	6458	-71.8	4608	1532	200.8
1050	2979	7557	-60.6	5132	1868	174.7
1100	4079	8445	-51.7	5878	2466	138.4
1150	5521	9532	-42.1	6445	2880	123.8
1200	6681	10394	-35.7	7214	3518	105.1

The results for the four emissions measures of effectiveness are presented in Table 7.8. It can be seen that the total emissions are reduced with the smaller gap acceptance parameters in all but two cases (total CO emissions, MRV 1150-1200 vph). The values presented in Table 7.8 are shown plotted in Figures 7.11 through 7.14.

TABLE 7.8 Roundabout Total Emissions

MRV	*Total HC emissions (kg/h)*			*Total CO emissions (kg/h)*			*Total CO_2 emissions (kg/h)*			*Total NO_X emissions (kg/h)*		
(vph)	*Md/De*	*SIDRA*	*% change*	*Md/De*	*SIDRA*	*% change*	*Md/De*	*SIDRA*	*% change*	*Md/De*	*SIDRA*	*% change*
50	0.039	0.039	0.0	1.53	1.54	-0.6	30.90	31.00	-0.3	0.058	0.059	-1.7
100	0.076	0.077	-1.3	3.05	3.09	-1.3	60.60	60.90	-0.5	0.116	0.117	-0.9
150	0.118	0.120	-1.7	4.79	4.86	-1.4	93.70	94.20	-0.5	0.180	0.182	-1.1
200	0.159	0.161	-1.2	6.49	6.61	-1.8	125.30	126.10	-0.6	0.242	0.245	-1.2
250	0.201	0.204	-1.5	8.27	8.43	-1.9	157.70	158.70	-0.6	0.306	0.310	-1.3
300	0.243	0.246	-1.2	10.07	10.27	-1.9	189.40	190.70	-0.7	0.370	0.375	-1.3
350	0.288	0.292	-1.4	12.01	12.26	-2.0	223.00	224.60	-0.7	0.439	0.445	-1.3
400	0.334	0.338	-1.2	14.02	14.34	-2.2	257.20	259.20	-0.8	0.509	0.517	-1.5
450	0.381	0.387	-1.6	16.13	16.52	-2.4	291.90	294.40	-0.8	0.582	0.591	-1.5
500	0.429	0.436	-1.6	18.30	18.77	-2.5	326.60	329.70	-0.9	0.655	0.667	-1.8
550	0.479	0.488	-1.8	20.61	21.19	-2.7	362.70	366.60	-1.1	0.733	0.747	-1.9
600	0.531	0.549	-3.3	23.04	24.08	-4.3	399.60	407.50	-1.9	0.814	0.838	-2.9
650	0.590	0.629	-6.2	25.74	27.77	-7.3	439.30	456.50	-3.8	0.900	0.947	-5.0
700	0.655	0.720	-9.0	28.59	31.13	-8.2	481.40	509.90	-5.6	0.989	1.048	-5.6
750	0.757	0.900	-15.9	32.72	37.02	-11.6	539.70	603.90	-10.6	1.108	1.208	-8.3
800	0.925	1.250	-26.0	37.68	47.51	-20.7	627.50	774.10	-18.9	1.248	1.479	-15.6
850	1.261	1.744	-27.7	45.14	61.08	-26.1	790.10	1011.00	-21.8	1.455	1.828	-20.4
900	1.696	2.300	-26.3	52.93	75.03	-29.5	995.60	1276.20	-22.0	1.679	2.191	-23.4
950	2.302	2.990	-23.0	61.83	90.41	-31.6	1279.00	1602.80	-20.2	1.948	2.598	-25.0
1000	3.261	3.691	-11.6	76.74	105.24	-27.1	1720.10	1933.70	-11.0	2.380	2.994	-20.5
1050	4.123	4.416	-6.6	95.79	119.77	-20.0	2118.70	2276.40	-6.9	2.879	3.388	-15.0
1100	5.205	5.168	0.7	115.66	134.79	-14.2	2617.60	2630.60	-0.5	3.424	3.794	-9.8
1150	6.380	6.010	6.2	137.86	150.24	-8.2	3159.40	3027.50	4.4	4.026	4.221	-4.6
1200	7.745	6.851	13.0	158.70	165.51	-4.1	3786.60	3422.00	10.7	4.626	4.643	-0.4

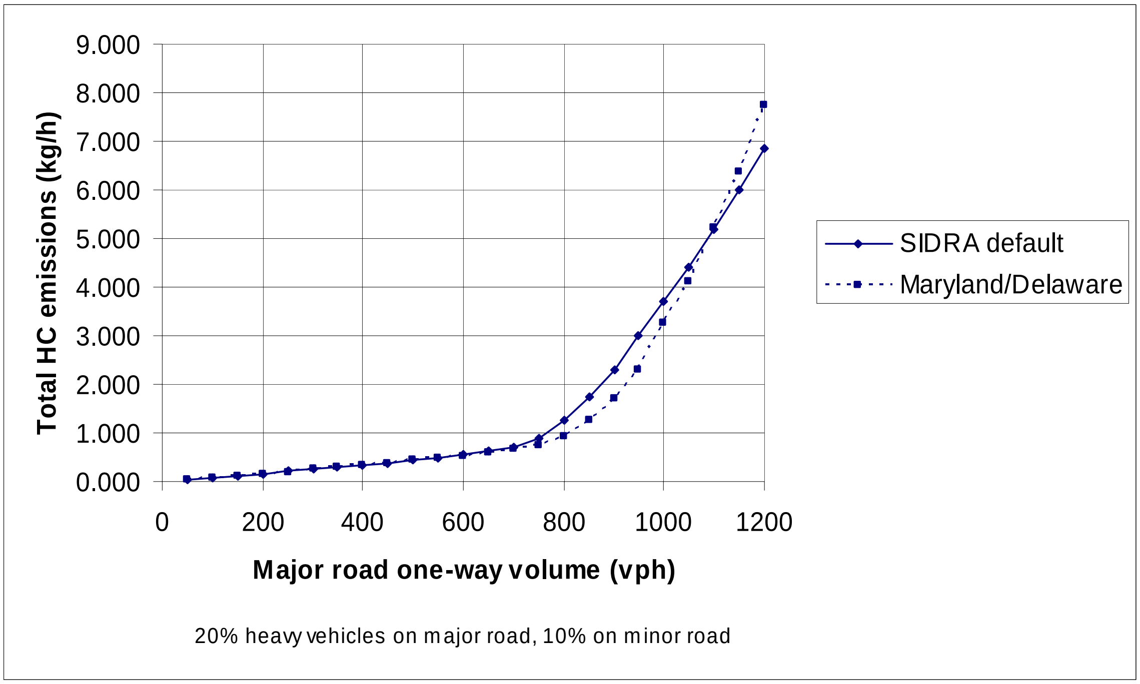


FIGURE 7.11 Roundabout Total HC Emissions

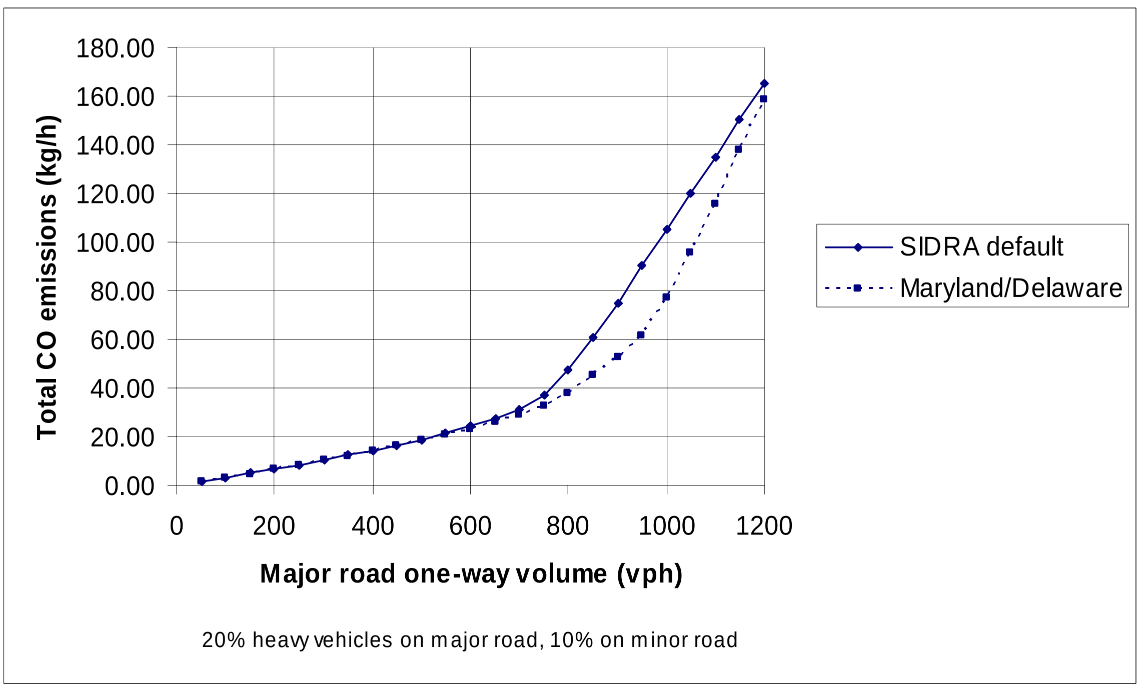


FIGURE 7.12 Roundabout Total CO Emissions

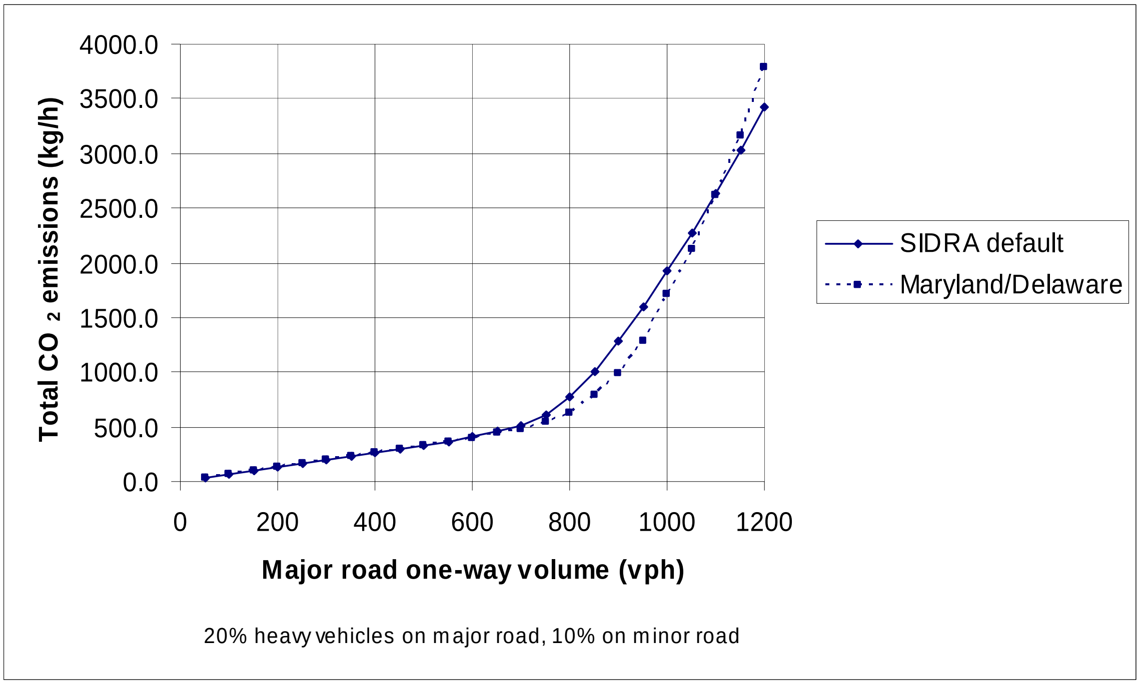


FIGURE 7.13 Roundabout Total CO_2 Emissions

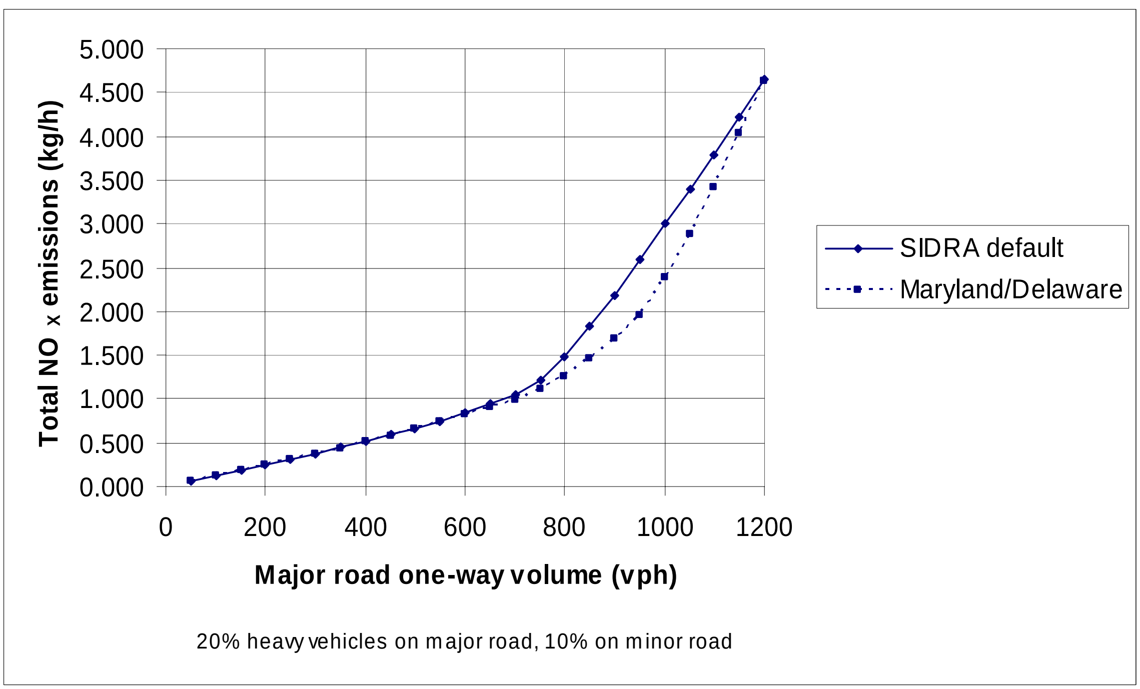


FIGURE 7.14 Roundabout Total NO_X Emissions

7.3 Changes in Threshold Volumes

The decreased critical gap and follow-up times observed from data collection generally created an increase in capacity and a decrease in delay, queue length, and emissions. These smaller gap acceptance parameters also led to an increase in the range of major road one-way volumes for which a roundabout may be more appropriate than a two-way stop controlled or signalized intersection. Table 7.9 summarized the volumes determined from the analysis as the threshold volumes between intersection types for the observed Maryland/Delaware and SIDRA default gap acceptance parameters and in both heavy vehicle scenarios. It can be seen that the smaller critical gap and follow-up times decreased the TWSC-Roundabout threshold volume and increased the Roundabout-Signal threshold volume for all three cases considered.

TABLE 7.9 Changes in Threshold Volumes between Intersection Types for Observed Maryland/Delaware and SIDRA Default Gap Acceptance Parameters

HV percentage		*Threshold based upon*	*TWSC-Roundabout*		*Roundabout-Signal*	
Major Rd.	*Minor Rd.*		*MD/DE*	*SIDRA*	*MD/DE*	*SIDRA*
10	10	Effective Intersection Capacity	0	115	809	680
		Average of Emissions	282	306	859	670
		Weighted Average of all MOEs	150	216	855	705
20	10	Effective Intersection Capacity	0	114	761	623
		Average of Emissions	258	314	769	594
		Weighted Average of all MOEs	137	190	840	669

The values listed in Table 7.9 indicate the lower and upper limits of ranges for which roundabouts are best suited, based upon the indicated criteria. A graphical representation of these ranges is presented in Figure 7.15, where the changes between varying criteria and heavy vehicle percentages are more readily seen. The smaller critical gap and follow-up time observed in data collection increased the MRV range for roundabouts an average of 54% over the range obtained using the SIDRA software default values. Increasing the heavy vehicle percentage on the main road from 10% to 20% decreased the MRV range for roundabouts an average of 9%.

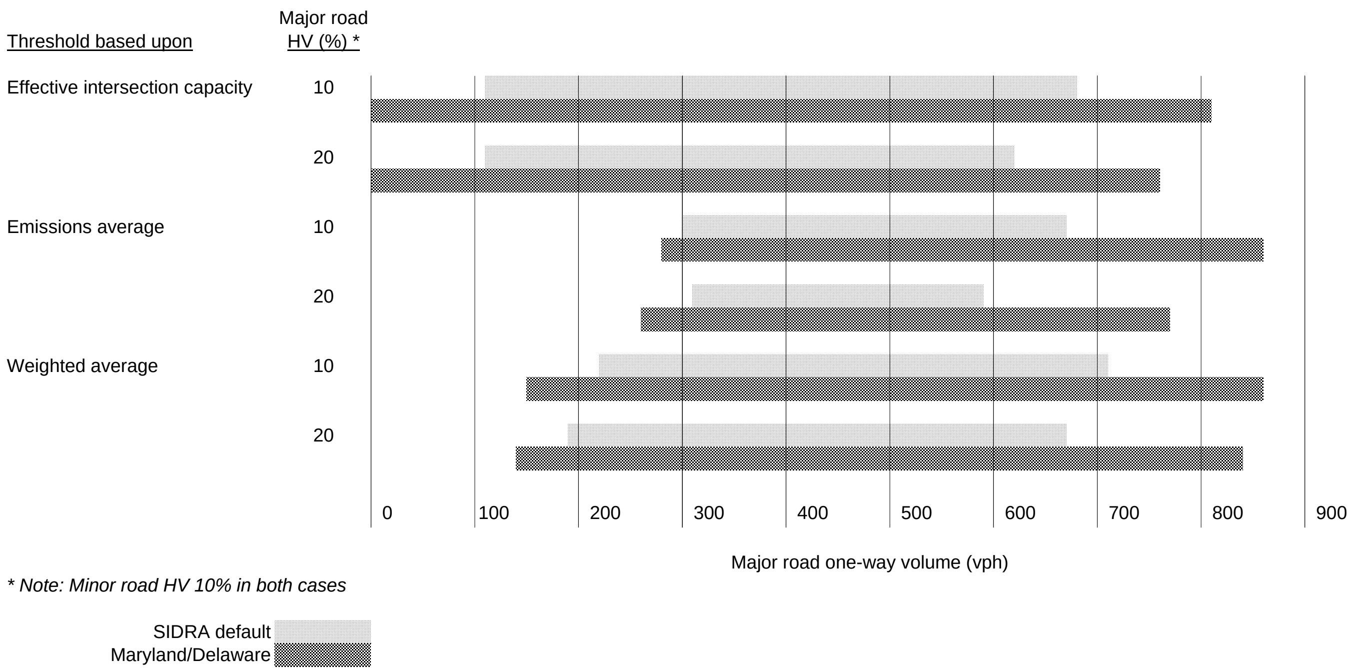


FIGURE 7.15 MRV Ranges for which Roundabouts are Most Appropriate Using Observed Maryland/Delaware and SIDRA Default Gap Acceptance Parameters

8.0 ROUNDABOUT VOLUME THRESHOLDS

Among the highlights of this study is the development of major road one-way volume thresholds which help define volume conditions under which each of the three intersection types (two-way stop-controlled, single-lane roundabout, and pre-timed signal) are most appropriate based upon the MOEs considered in this study.

Figures 8.1 through 8.4 are graphical representations of Tables 5.1, 5.2, 6.1, 6.2, and 7.9. The heavy lines represent two possible ways of determining overall threshold volumes, one based solely upon the effective intersection capacity and the other on a weighted average of all MOEs. The weighted average was determined by assigning weights as follows: effective intersection capacity (3), major road capacity (1), minor road capacity (1), major road average delay (2), minor road average delay (2), major road 95% queue (1), minor road 95% queue (1), total HC emissions (2), total CO emissions (2), total NO_X emissions (2), and total CO_2 emissions (2). These weights were determined while keeping in mind the focus of this study, which was the consideration of vehicular emissions.

Figures 8.1 and 8.2 present the volume ranges based upon the effective intersection capacity and the weighted average using the SIDRA default gap acceptance parameters for each of the heavy vehicle scenarios. Figures 8.3 and 8.4 show thresholds based upon the same conditions using the critical gap and follow-up times determined from data collection in Maryland for each of the heavy vehicle scenarios. Figures 8.1 through 8.4 all show the same occurrence where the MRV range for roundabouts is shifted to the right when the weighted average of all MOEs is considered rather than solely the effective intersection capacity.

The decreased critical gap and follow-up times obtained from data collection generally resulted in an increase in capacity and a decrease in delay, queue length, and emissions. These smaller gap acceptance parameters also led to an increase in the range of major road one-way volumes for which a roundabout may be more appropriate than a two-way stop controlled or signalized intersection. Comparing Figures 8.1 and 8.2 with Figures 8.3 and 8.4, it can be seen that the lesser critical gap and follow-up times **decreased** the TWSC-Roundabout threshold volume and **increased** the Roundabout-Signal threshold volume for effective intersection capacity and weighted average criteria. As discussed previously in Chapter 7, the smaller critical gap and follow-up time observed in data collection increased the MRV range for roundabouts an average of 54% over the range obtained using the SIDRA software default values. The increase in heavy vehicle percentage on the main road from 10% to 20% decreased the MRV range for roundabouts an average of 9%.

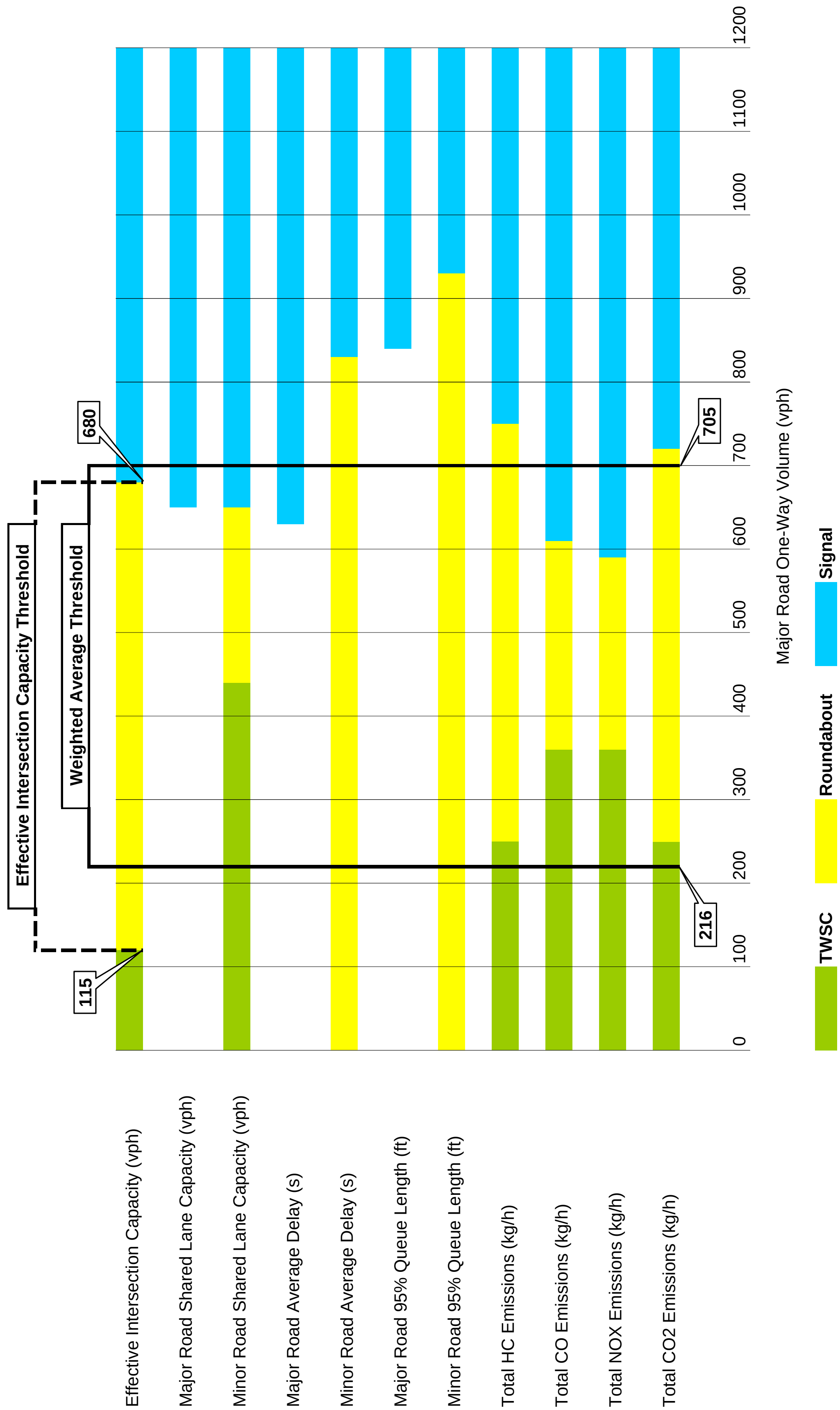


FIGURE 8.1 Volume Thresholds between Intersection Types using SIDRA Default Gap Values
10% heavy vehicles on both roads

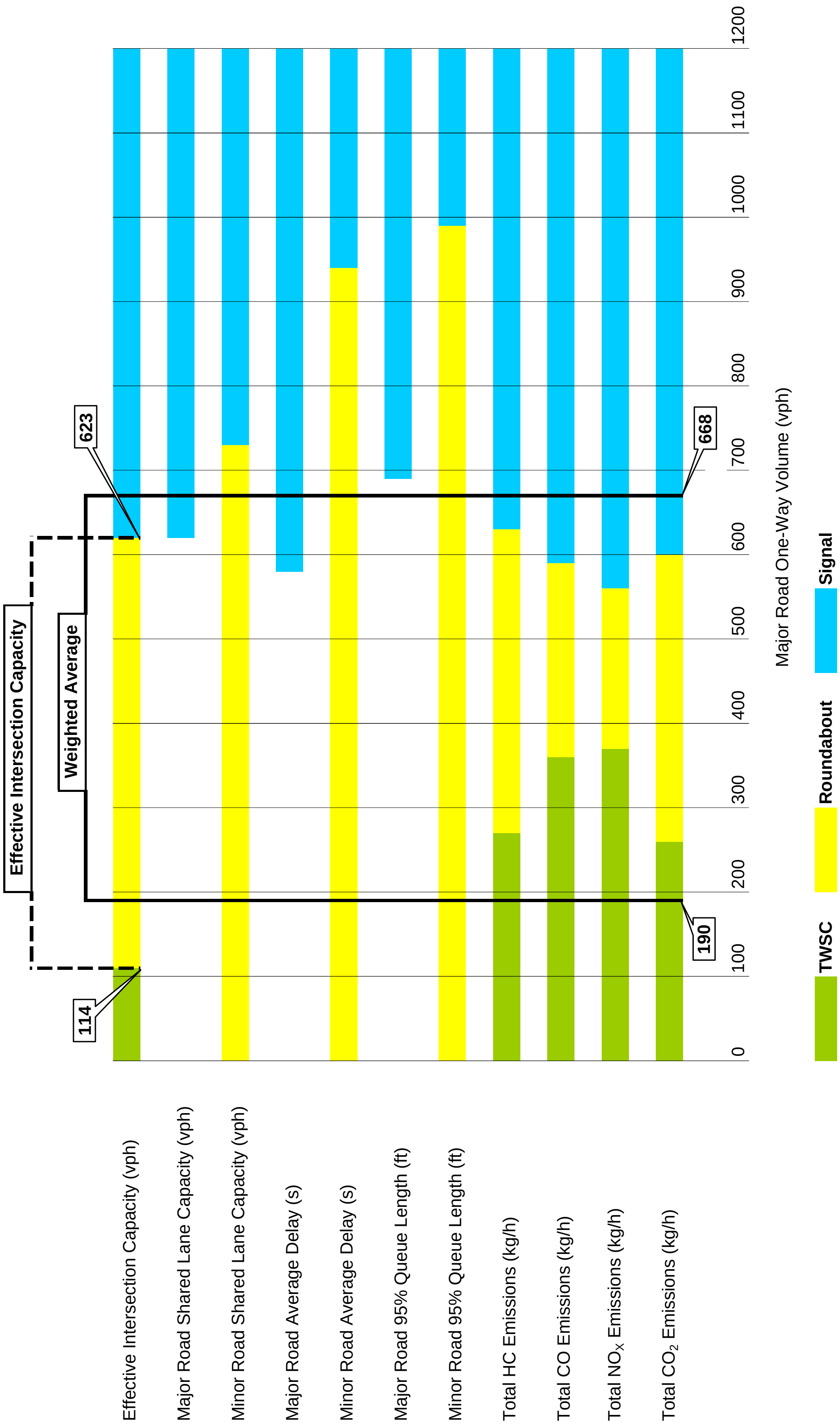


FIGURE 8.2 Volume Thresholds between Intersection Types using SIDRA Default Gap Values
20% heavy vehicles on major road, 10% on minor road

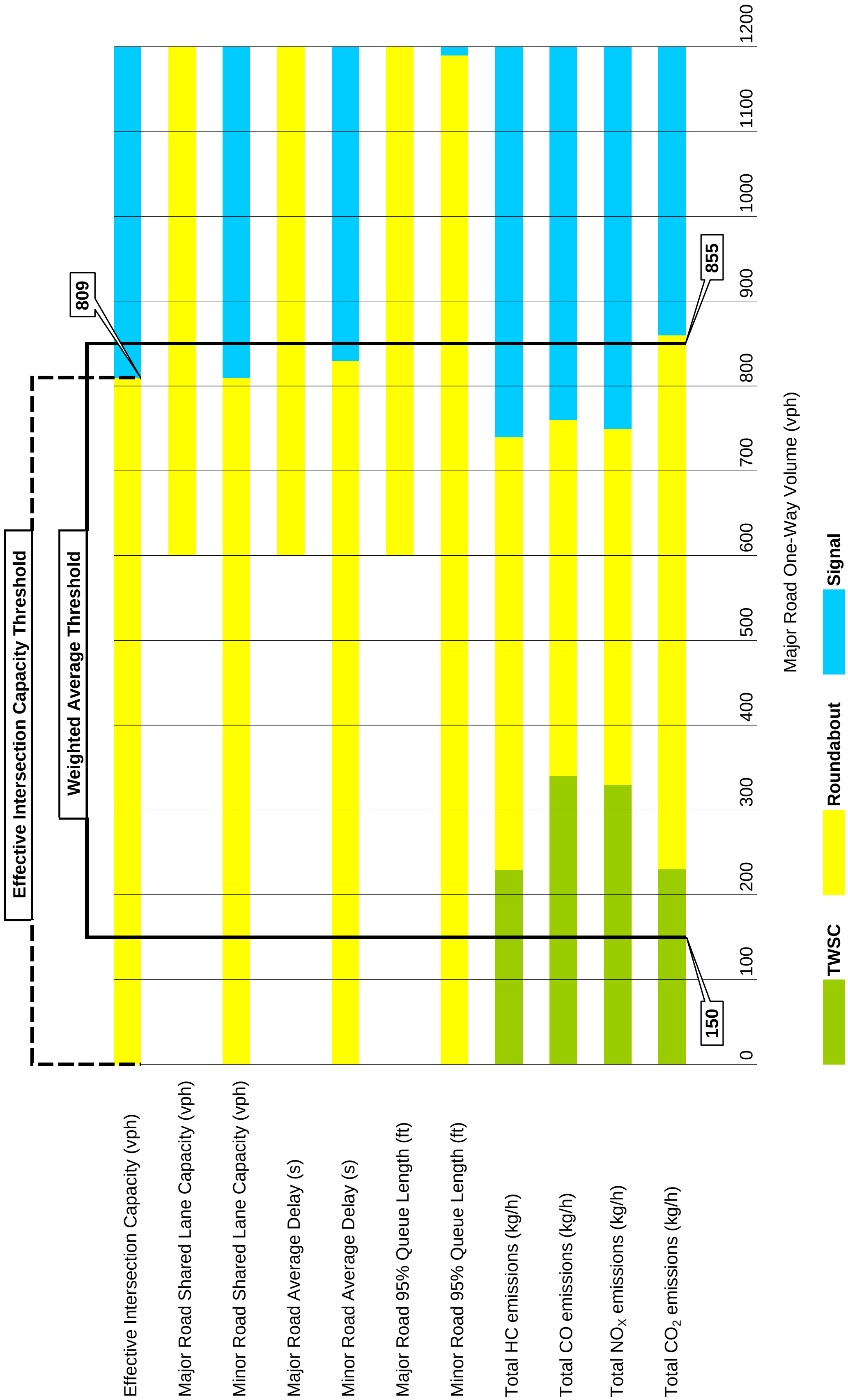


FIGURE 8.3 Volume Thresholds between Intersection Types using Maryland/Delaware Gap Values
10% heavy vehicles on both roads

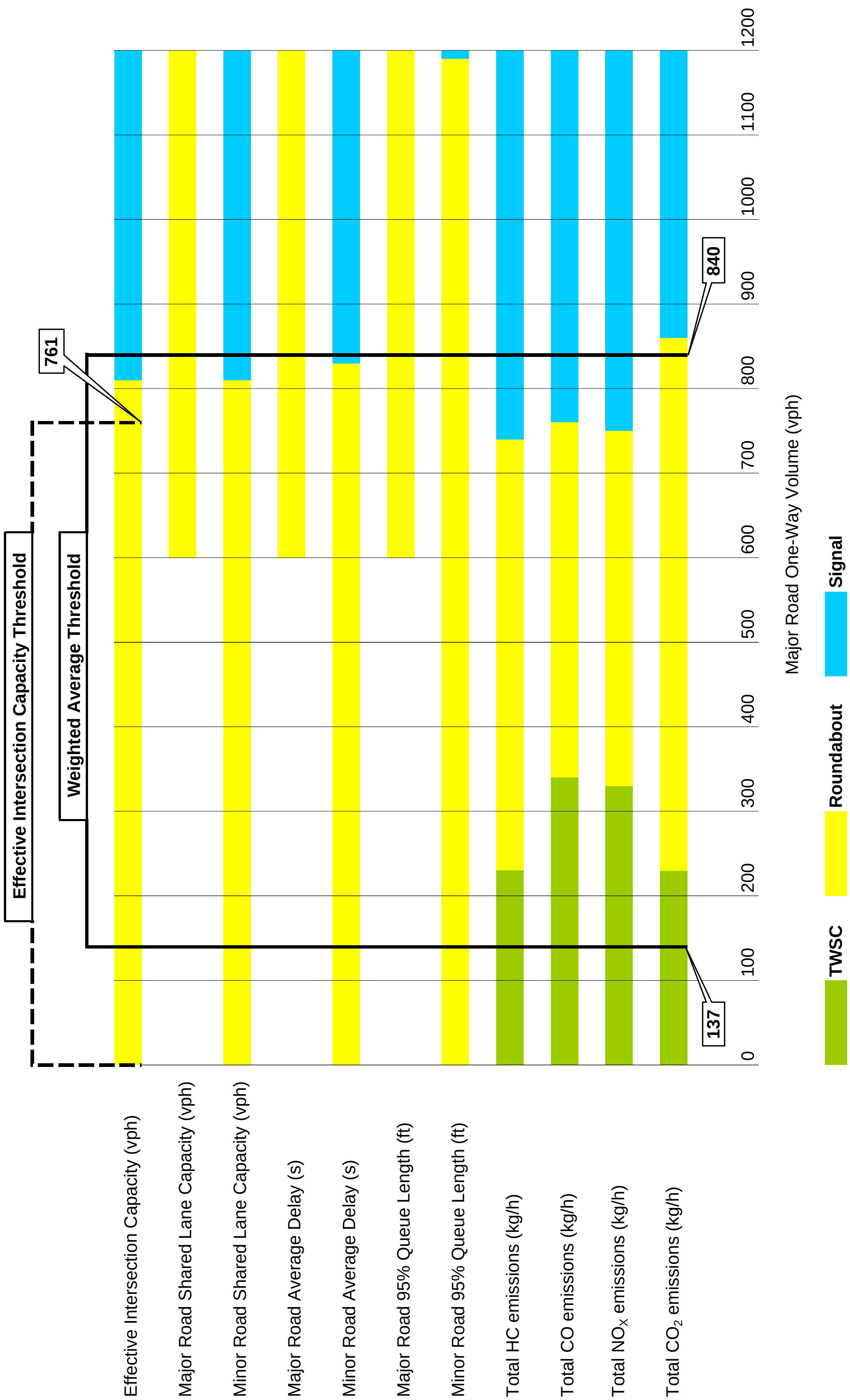


FIGURE 8.4 Volume Thresholds between Intersection Types using Maryland/Delaware Gap Values
20% heavy vehicles on major road, 10% on minor road

9.0 CONCLUSIONS

This study analyzed three intersection types: two-way stop controlled, roundabout, and pretimed signal, and compared them with one another based on several performance measures of effectiveness provided by the SIDRA simulation software. The selected MOEs were: effective intersection capacity (entire intersection), main road capacity, minor road capacity, major road average delay, minor road average delay, major road 95% queue length, minor road 95% queue length, and four vehicular emission levels (CO, CO_2, HC, and NO_X).

Comparisons were made between the unsignalized intersection and roundabout at lower major road one-way volumes (MRV) and between the signalized intersection and roundabout at higher MRV. The analysis identified the MRV at which the advantage switched between intersection types for each MOE considered. In order to obtain general ranges for which each intersection type performs best, a set of overall volume thresholds were determined based upon the effective intersection capacity, as well a set of thresholds based upon a weighted average of the MOEs considered. This was first conducted using the SIDRA software default values and repeated a second set of analyses using critical gap and follow-up times determined from data collection at two roundabouts located in Maryland near its border with Delaware.

In order to obtain gap acceptance characteristics more closely resembling those of Delaware drivers, two nearby roundabouts were observed. Due to the lack of roundabouts in Delaware experiencing volumes sufficient for obtaining critical gap and follow-up times of drivers, two locations in eastern Maryland were chosen for observation. These sites are the intersections of MD 273 and 276 in Cecil County and MD 18 and Castle Marina Road in Queen Anne's County. The videotapes were reviewed and critical gap values of 3.85 and 3.91 s were found for the study sites, which are lower than those values of 4.1 to 4.6 s recommended by the HCM 2000. The follow-up times found for the sites are 2.1 and 2.3 s compared to 2.6 to 3.1 provided by the HCM 2000.

In general, it was found that the two-way stop controlled intersection performed best for relatively low major road one-way volumes, the pretimed signal performed best for relatively high major road one-way volumes, and the roundabout performed best for a mid-range volume between the two.

Preliminary analyses using the SIDRA software default gap values showed that the approximate MRV range for which a single-lane roundabout is deemed most reasonable for use at an intersection with the geometric and traffic conditions assumed in this study is between approximately 220 and 700 vph (see Figure 8.1). When heavy vehicles on the major road are increased, this range becomes approximately 190 to 670 vph (see Figure 8.2). Follow-up analyses using the SIDRA software calibrated for Maryland/Delaware drivers showed that roundabouts are best suited for an MRV range of approximately 150 to 850 vph (see Figure 8.3). When heavy vehicles along the major road are increased, the suitable range becomes approximately 140 to 840 vph (see Figure 8.4).

Further research related to this study is suggested on several topics: (1) an analysis of the volume ranges suitable for the installation of a roundabout instead of a four-way stop-controlled conventional intersection, (2) the impact of a greater range of heavy vehicle percentages on the proposed volume thresholds, and (3) the sensitivity of the proposed thresholds to critical gap and follow-up times so that specific volume thresholds could be applied to other locations with different driver gap acceptance characteristics.

In conclusion, our recommendation for the range of major road one-way volumes for which the construction of a roundabout may be preferred to that of a two-way stop-controlled intersection or a pre-timed signalized intersection is summarized below. Figure 9.1 shows volume ranges for both heavy vehicle scenarios on the major road, and is based on SIDRA analyses using the Maryland/Delaware gap acceptance parameters collected for this study.

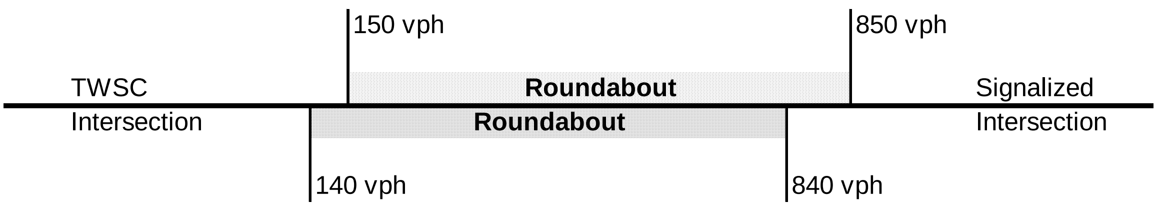


FIGURE 9.1 Recommended Ranges of Major Road One-way Volumes for which the Construction of Roundabouts is Preferred to that of TWSC or Signalized Intersections. 10% Heavy Vehicles on the Major Road (top), 20% Heavy Vehicles on the Minor Road (bottom)

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