Capacity and Performance Analysis of Roundabout Metering Signals

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ABSTRACT

This paper describes a method for the analysis of capacity and performance of roundabouts operating with metering signals. When low capacity conditions occur during peak demand flow periods, for example due to unbalanced flow patterns, the use of metering signals is a cost-effective measure to avoid the need for a fully-signalized intersection treatment. Roundabout metering signals are often installed on selected roundabout approaches and used on a part-time basis since they are required only when heavy demand conditions occur during peak periods. Metering signals have been used in Australia, UK and USA to alleviate the problem of excessive delay and queuing by creating gaps in the circulating stream. The Australian roundabout and traffic signal guides acknowledge the problem and discuss the use of metering signals. The basic principles of the operation of roundabout metering signals are explained. Case studies of various roundabouts where metering signals were used, or considered for use, have been presented in previous papers by the author. This paper presents the results of analysis of one of these case studies when operating with metering signals.

INTRODUCTION

Implementation and continued success of modern roundabouts in the USA, as in many countries around the world, depend on improved understanding of major factors that affect the operation of roundabouts. Like all other traffic control devices, the road and intersection geometry, driver behavior, light and heavy vehicle characteristics, behavior and requirements of other road users, traffic flow characteristics and operation of traffic control to resolve vehicle to vehicle conflicts (as well as vehicle to pedestrian conflicts) are important factors that influence roundabout performance. Vehicle traffic flow characteristics represent collective behavior of vehicles in a traffic stream as relevant to, for example, car following, queue forming and queue discharge conditions.

The control rule at modern roundabouts is the yield (give-way) rule. Analytical and microsimulation models use gap-acceptance modeling to emulate behavior of entering drivers yielding to circulating vehicles, i.e. finding a safe gap (headway) before entering a roundabout. This behavior is affected by roundabout geometry (size, entry and circulating lane widths, approach and circulating lane arrangements, etc.) which influences such important parameters as sight distance, speed and lane use. The headway distribution of vehicles in the circulating stream (influenced by queuing on the approach road and effective use of circulating lanes at multi-lane roundabouts) is the controlling variable that determines the ability of approach vehicles to enter the circulating road. This is as important as the critical gap (headway) and follow-up headway parameters of the entry stream in determining roundabout capacity, performance (delay, queue length, number of stop-starts, fuel consumption, emissions, and operating cost) and level of service.

Thus, complex interactions among the geometry, driver behavior, traffic stream and control factors determine the roundabout capacity and performance. The level of traffic performance itself can influence driver behavior, increasing the complexity of modeling roundabout operations.

The operation of a roundabout is a *closed-loop system* where the conditions of traffic streams entering from approach roads affect traffic on other approaches. As a result, an important factor that influences the capacity and performance of traffic on roundabout approach roads is the *origin-destination pattern* of arrival (demand) flows as related to the approach and circulating lane use. This impacts headway distributions of circulating streams, and as a result, affects approach lane capacities and performance. The origin-destination factor and the related issues of *priority reversal* and *priority emphasis* are discussed in some detail due to their relevance to roundabout operating conditions that require metering signals.

When low capacity conditions occur during peak demand flow periods, for example due to unbalanced flow patterns *(1,2)*, the use of metering signals is a cost-effective measure to avoid the need for a fully-signalized intersection treatment. Roundabout metering signals are often installed on selected roundabout approaches and used on a part-time basis since they are required only when heavy demand conditions occur during peak periods. Metering signals have been used in Australia, UK and USA to alleviate the problem of excessive delay and queuing by creating gaps in the circulating stream. The Australian roundabout and traffic signal guides acknowledge the problem and discuss the use of metering signals *(3,4)*. The basic principles of the operation of roundabout metering signals are explained.

Case studies of various roundabouts where metering signals were used, or could be considered for use, have been presented in a recent paper by the author *(1)*. These case studies included one-lane, two-lane and three-lane roundabouts from Australia, UK and the USA with total intersection flow rates in the range 1700 to 5300 veh/h. These case studies also demonstrated the importance of modeling different approach and circulating lane arrangements at multi-lane roundabouts. This paper presents detailed results of an analysis of a case of roundabout operating with metering signals based on a case discussed in previous papers. The aaSIDRA 2.1 micro-analytical software package was used for this analysis *(5-7)*.

aaSIDRA employs an *empirical* gap-acceptance method to model roundabout capacity and performance. The model allows for the effects of both roundabout *geometry* and *driver behaviou*r, and it incorporates effects of *priority reversal* (low critical gaps at high circulating flows), *priority emphasis* (unbalanced O-D patterns), and *unequal lane use* (both approach and circulating lanes). Capacity can be measured as a *service rate* for each traffic *lane* in undersaturated conditions (v/c ratios less than 1) according to the HCM definition of capacity to represent *prevailing conditions*. This is in contrast with measuring *approach capacity* in oversaturated conditions.

THE ORIGIN-DESTINATION (O-D) FACTOR

The O-D factor was first introduced in an earlier version of aaSIDRA to allow for unbalanced flow effects after research was conducted *(8-14)* following reports received from many practitioners that overoptimistic results were obtained using the Australian (AUSTROADS) Roundabout Guide method *(3)*. The O-D factor method represents a substantial change to the method described in the AUSTROADS Roundabout Guide from which aaSIDRA originated *(6)*.

The aaSIDRA model contrasts with other methods that treat the roundabout as *a series of independent T-junctions* with no interactions among approach flows (except that some traditional methods allow for the effect of capacity constraint on circulating flows). While traditional methods may be adequate for low flow conditions, the O-D factor improves the prediction of capacities under medium to heavy flow conditions, especially with *unbalanced* demand flows. This helps to avoid capacity overestimation under such conditions as observed at many real-life intersections. The case studies reported previously and the case analyzed in this paper are examples of such cases. In all real-life cases considered, the methods without unbalanced flow modeling predict good operating conditions whereas long delays and queues are observed on one or more approaches of such roundabouts.

Figure 1 explains the effect of the O-D pattern. It can be seen that different capacities and levels of performance may be estimated for the same circulating flow rate depending on the conditions of the component streams. The lowest capacity is obtained when the component stream flow rates are unbalanced and the main (dominant) stream is a very large proportion of the total circulating flow, it is in a single lane, and is highly queued on the approach lane it originates from.

Generally, the extent of the unbalanced flow problem is likely to be underestimated by the TRL (UK) linear regression model, HCM 2000 and AUSTROADS gap-acceptance models, and similar models that:

(i) estimate low capacity for approaches with high entry flows against low circulating flows, and

(ii) do not have sensitivity to the origin-destination pattern.

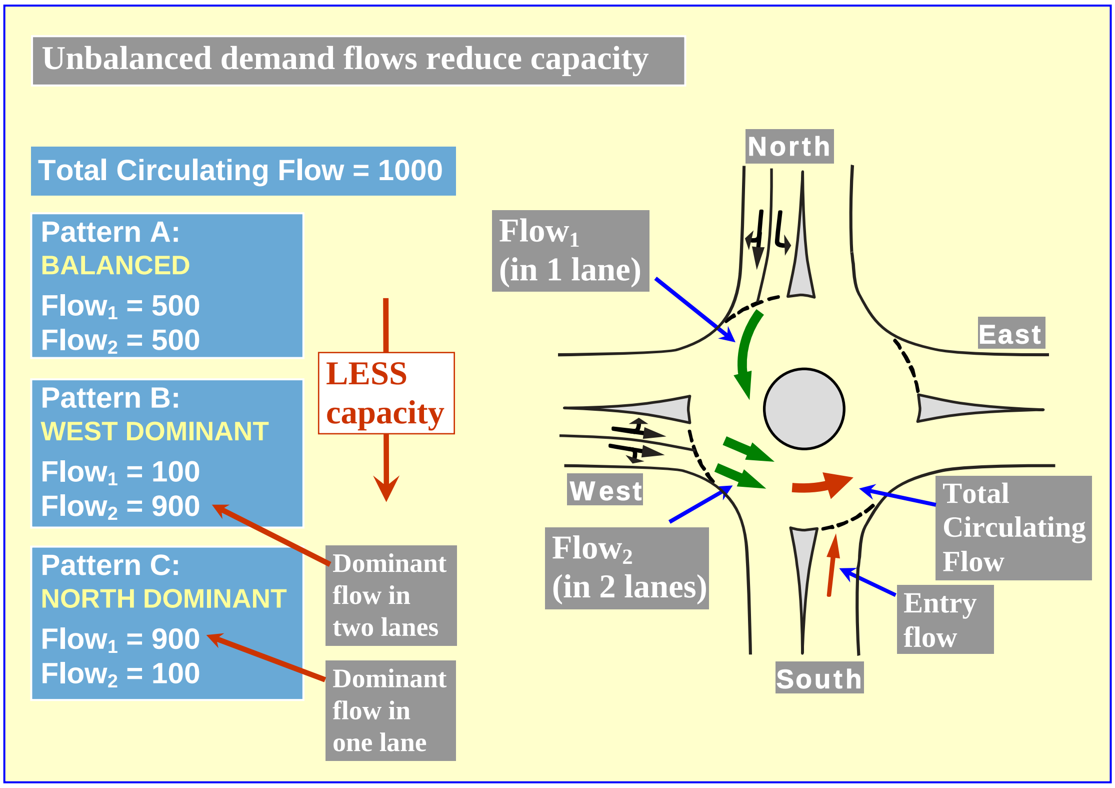

Figure 1 - The effect of the Origin-Destination (O-D) pattern on capacity in modeling unbalanced flows

The level of capacity overestimation at the downstream approach will increase when the upstream approach is estimated to be oversaturated, in which case, *capacity constraint* would be applied to the upstream approach. Capacity constraint means that if the arrival (demand) flow on an approach exceeds capacity, only the capacity flow rate is allowed to enter the roundabout circulating road. This would lead to an unrealistically low circulating flow in front of the downstream approach, and therefore to an increased capacity estimate for the downstream approach.

PRIORITY SHARING AND PRIORITY EMPHASIS

The limited-priority method of gap-acceptance modeling described by Troutbeck and Kako *(15-17)* allows for priority sharing between entering and circulating vehicles in order to introduce a correction to the gap-acceptance capacity formula based on absolute priority of circulating stream vehicles. The need for adjustment is due to low critical gap values at high circulating flow rates which may result in the condition "follow-up headway + intra-bunch (minimum) headway > critical gap (headway)". The limited-priority method *reduces* the capacity estimated by the absolute-priority method.

The O-D factor used in the aaSIDRA roundabout capacity model incorporates the effect of priority sharing in adjusting the roundabout capacity function. Furthermore, the non-linear relationship between the critical gap and circulating flow rate used in aaSIDRA version 2.1 reduces the amount of adjustment to the capacity function based on absolute priority since it estimates larger critical gap values at high circulating flows, unlike the linear model in the AUSTROADS Roundabout Guide model *(3)*.

Roundabout circulating streams are *uninterrupted* flows in short segments on the circulating road (between *entry - circulating road junctions*), and they contain *queued vehicles* entering from approach lanes. The O-D factor allows for the fact that vehicles departing from approach queues with follow-up (saturation) headways are under *forced flow* conditions, and as such they are considered to be *bunched*.

Without the O-D factor, which in effect, modifies the circulating stream headway distribution model, the gap-acceptance capacity formula gives unduly high capacity estimates at medium to high circulating flow rates, especially for multilane roundabouts.

While the O-D factor allows for capacity reduction needed to model priority sharing, it also allows for reduced unblock time due to an opposite effect, which can be called *priority emphasis.*

The priority emphasis condition occurs in the case of unbalanced flow patterns when a dominant flow restricts the amount of entering traffic since most vehicles in the circulating stream have entered from a queue at the upstream approach continuously due to a low circulating flow rate against them (see *Figure 2*). Even a small amount of circulating flow can cause a significant proportion of vehicles to be queued on an approach with a heavy flow rate, although the capacity can be high. This also corresponds to the case of long back of queue and low delay.

A heavy stream that can enter the roundabout with little interruption due to a low circulating flow rate against it (*unbalanced flow* conditions) represents mainly forced flow conditions, and cause reduced capacity at a downstream entry. The *origin-destination factor* in aaSIDRA takes into account the flow balance as well as the amount of queuing in the circulating stream, in effect modifying the circulating stream headway distribution to allow for these factors.

Without allowance for *priority emphasis*, any method based on gap-acceptance modeling with or without limited-priority process, or any comparable empirical method, fails to provide satisfactory estimates of roundabout capacity with unbalanced flows.

Figure 2 - An example of dominant entry flow at a modern roundabout:
Grange Rd, St Georges Rd and Alexandra Avenue in Toorak, Melbourne, Australia
(photo modified for driving on the right-hand side of the road)

UNBALANCED FLOWS AT ROUNDABOUTS - THE ISSUE

Improved understanding of the effect of the origin-destination pattern of traffic on roundabout capacity, performance and level of service helps towards designing new roundabouts that will cope with future increases in demand levels and solving any problems resulting from unbalanced flow patterns at existing roundabouts. Many real-life case studies show that roundabout capacity and level of service depend not only on the circulating flow level but also the balance, queuing and lane use characteristics of approach flows contributing to the circulating flow *(1,2,11-14)*. Unbalanced flow conditions may arise at T-intersection, four-way and freeway interchange roundabouts.

Dominant circulating flows, originating mostly from a single approach, reduce the entry capacity, as evident from the use of metering signals or other types of signalization in Australia *(1-4,11-14)*, UK *(18-23)* and USA *(24)* to alleviate the problem of excessive delay and queuing by creating gaps in the circulating stream.

Huddart's *(18)* comments published as early as 1983 explains the issue clearly: "*…the proper operation of a roundabout depends on there being a reasonable balance between the entry flows. … an uninterrupted but not very intense stream of circulating traffic can effectively prevent much traffic from entering at a particular approach.*" and "*The capacity of roundabouts is particularly limited if traffic flows are unbalanced. This is particularly the case if one entry has very heavy flow and the entry immediately before it on the roundabout has light flow so that the heavy flow proceeds virtually uninterrupted. This produces continuous circulating traffic which therefore prevents traffic from entering on subsequent approaches.*"

At a roundabout with an unbalanced flow pattern, a traffic stream with a heavy flow rate enters the roundabout against a circulating stream with a low flow rate. Examples of high flow rates per lane at such roundabout cases from Melbourne, Australia are described below.

(i) Small to medium size single-lane roundabout at the intersection of Grange Rd, St Georges Rd and Alexandra Avenue in Toorak (see *Figure 2*): **1693 veh/h per lane** entering against a circulating flow rate of 67 veh/h has been reported *(25)*. Sum of entering and circulating flows is 1760 veh/h. The measured follow-up headway and critical gap values for this entry lane are 1.992 s and 2.423 s, respectively. The maximum capacity at zero circulating flow (corresponding to the follow-up headway) is 3600 / 1.992 = 1808 veh/h.

(ii) Small single-lane roundabout at the intersection of Stanhope Grove with Broadway in Camberwell (see *Figure 3*): **1524 veh/h per lane** entering against a circulating flow rate of 60 veh/h has been reported *(26)*. The sum of entering and circulating flows is 1584 veh/h.

(iii) Large multi-lane roundabout at the intersection of Mickleham Rd and Broadmeadows Rd in Westmeadows: **1397 veh/h per lane** against a circulating flow rate of 83 veh/h in am peak and **1501 veh/h per lane** against a circulating flow rate of 112 veh/h in pm peak. The sum of entering and circulating flows is 1480 veh/h in am peak and 1613 in pm peak. This case is described in the Australian Roundabout Guide *(3)* and used for the example for metering signal analysis given in this paper.

Several studies related to the issue of unbalanced flows at roundabouts have been reported in the literature *(27,28)*. A study of a roundabout in Denmark *(28)* concluded that "the lane allocation of circulating flow did have a significant impact on capacity, particularly at large circulating flow rates. This implies that the origin and destination of the flow constituting the circulating traffic must be accounted for in estimating capacity."

Unbalanced flows may not be a problem when the overall demand level is low but the problem appears with traffic growth even at medium demand levels. Demand flow patterns and demand levels may change significantly after the introduction of a roundabout, sometimes in a relatively short period of time, because there is no direct control over turning movements unlike signalized intersections.

Figure 3 - Stanhope Grove with Broadway Roundabout: Camberwell, Melbourne, Australia *(26).*
(photo modified for driving on the right-hand side of the road)

Modeling of the traffic demand pattern is important in optimizing the roundabout geometry including lane arrangements. This can be achieved for a new roundabout subject to the reliability of traffic demand information, or for an existing roundabout to a smaller extent due to the design constraints imposed by existing geometry *(13)*. The use of part-time metering signals is a cost-effective measure to avoid the need for a fully-signalized intersection treatment. This is discussed below.

ROUNDABOUT METERING SIGNALS - A PRACTICAL SOLUTION TO THE UNBALANCED FLOW PROBLEM

There are many examples of roundabouts with unbalanced flow patterns in Australia, where part-time roundabout metering signals are used to create gaps in the circulating stream in order to solve the problem of excessive queuing and delays at approaches affected by highly directional flows *(1-4,11-14)*. The signalized roundabout solution has been used extensively in the UK as well *(18-23)*. A US paper discusses the use of metering signals for the Clearwater roundabout in Florida *(24)*. The Australian roundabout and traffic signal guides acknowledge the problem and discuss the use of metering signals *(3,4)*.

Roundabout metering signals are usually employed on a part-time basis since they may be required only when heavy demand conditions occur during peak periods. They can be an effective measure preventing the need for a fully-signalized intersection treatment as they are often used on selected roundabout approaches, operational only when needed under peak demand conditions.

Figure 4 shows typical arrangements for roundabout metering signals and an example from Melbourne, Australia (photo modified to show driving on the right-hand side of the road).

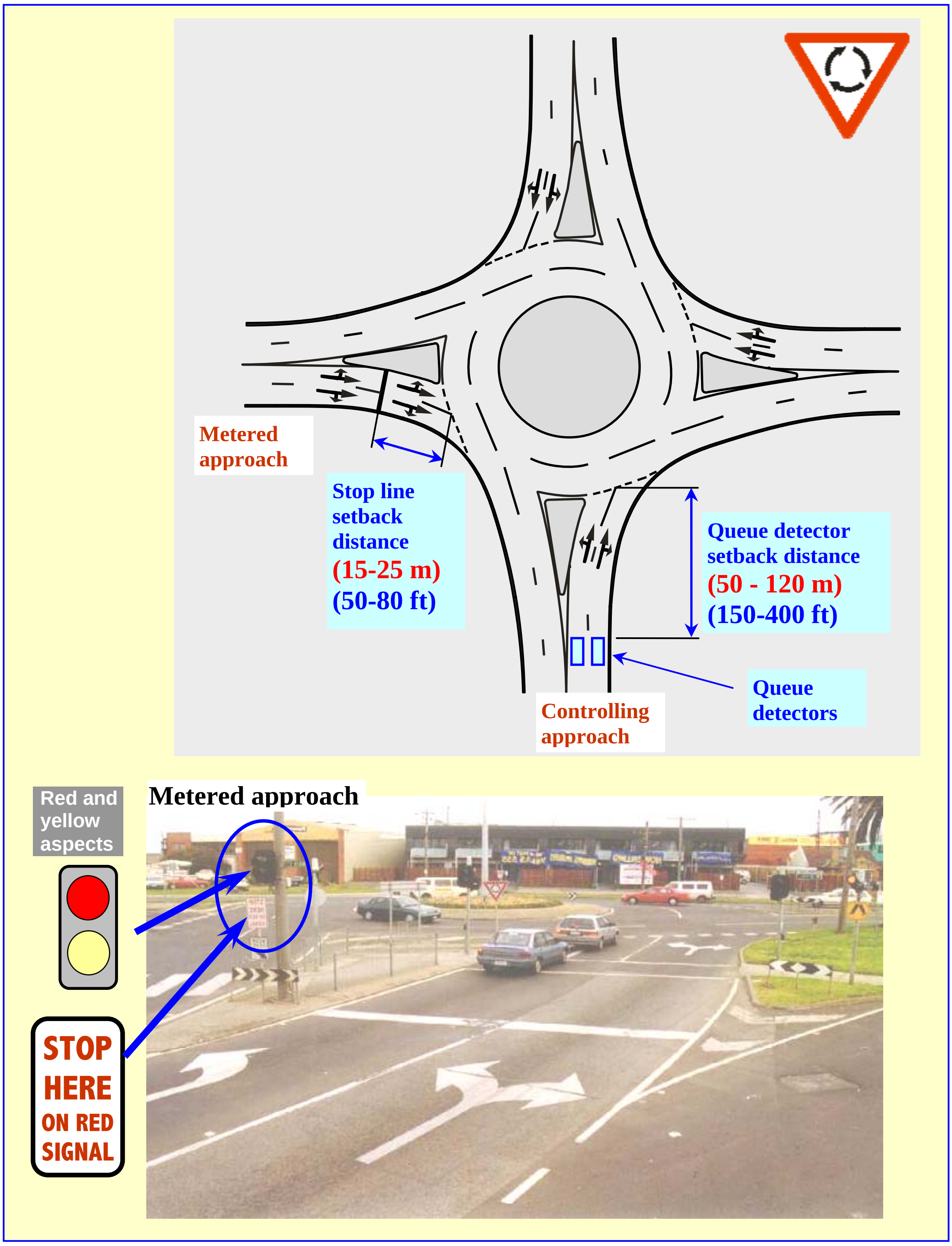

Figure 4 - Use of metering signals with an example from Melbourne, Australia
(photo modified to show driving on the right-hand side of the road)

As seen in *Figure 4*, the term *metered approach* is used for the approach stopped by red signals (approach causing problems for a downstream approach), and the term *controlling approach* is used for the approach with the queue detector, which is the approach helped by metering signals (approach experiencing problems due to a relatively heavy directional flow from the metered approach).

In *Figure 4*, two-aspect red and yellow signals are used where the sequence of aspect display is Off (Blank) to Yellow to Red to Off (Blank). When metering is not required neither aspect is displayed.

Various site-specific methods may also be used to meter traffic, e.g. using an existing upstream pedestrian-actuated signalized crossing on the metered approach as in *Figure 5*. In this case, three-aspect red, yellow and green signals are used. Three-aspect signals have been used for the metering signals at the Clearwater roundabout in Florida, USA as shown in *Figure 6* although these metering signals are used in a somewhat different context *(24)*.

The Australian Traffic Signal Guide *(4)* recommends the use of a minimum of two signal faces, one primary (signal face mounted on a post at or near the left of the stop line on the approach) and one tertiary (signal face mounted on a post on the downstream side to the left of that approach) for driving on the left-hand side of the road. A regulatory sign STOP HERE ON RED SIGNAL is fixed to any signal post erected adjacent to the stop line, as drivers do not expect to stop at the advance stop line location. Stop lines are located not less than 3 m in advance of the give-way (yield) line but are preferably positioned approximately 20 m (50 - 80 ft) from the give-way (yield) line. Queue detector setback distance on the controlling approach is usually in the range 50 m to 120 m (150 - 400 ft).

In some cases, it may be necessary to supplement the traffic signals with explanatory fixed or variable message signposting (for example, the sign "SIGNALS MAY BE CALLED BY …" in *Figure 5*). Where sight restrictions exist, advance warning signals are considered.

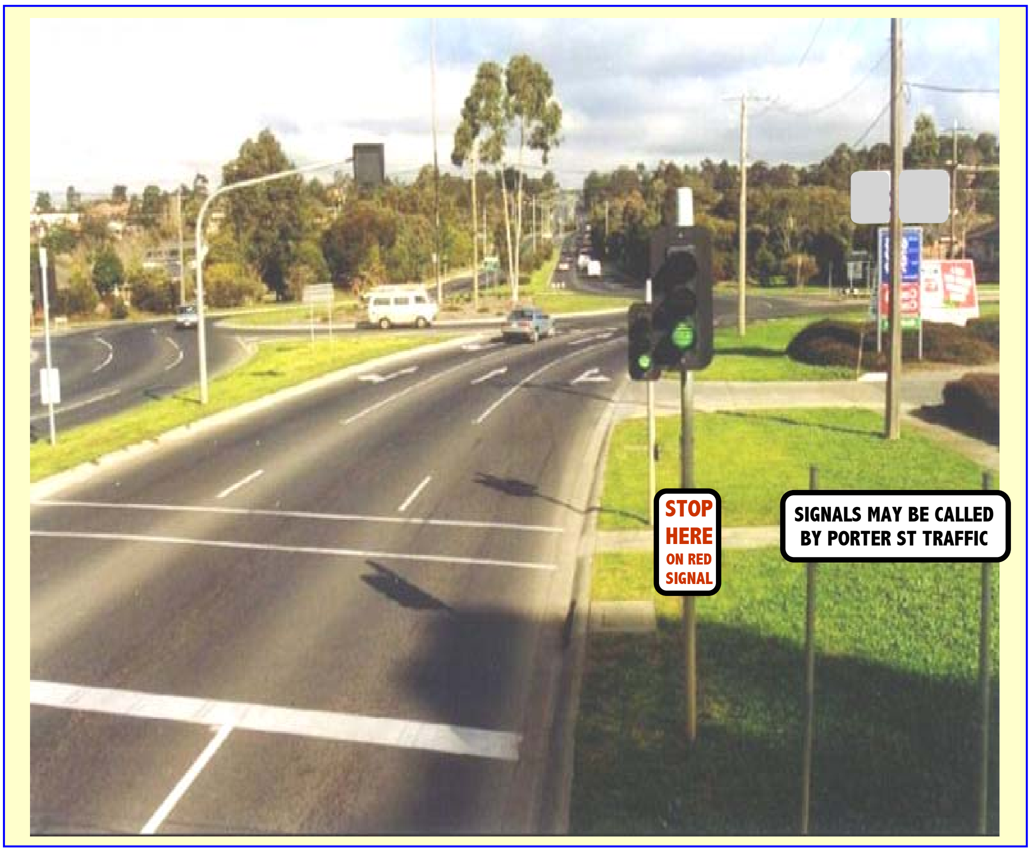

Figure 5 - Use of pedestrian-actuated signals for roundabout metering at Fitzsimons Lane - Porter St Roundabout, Melbourne, Australia (photo modified to show driving on the right-hand side of the road)

Figure 6 - Metering signals at the Clearwater roundabout, Florida, USA *(24)*

When the queue on the controlling approach extends back to the queue detector, the signals on the metered approach display red so as to create a gap in the circulating flow. This helps the controlling approach traffic to enter the roundabout. When the red display is terminated on the metered approach, the roundabout reverts to normal operation.

There are two types of operation depending on the use of detectors on the metered approach. Operation with no detectors on the metered approach is similar to the semi-actuated operation (metered approach corresponds to the major road, and the controlling approach corresponds to the side road). It is also similar to the operation of signalized pedestrian crossings with pedestrian actuation and no vehicle actuation.

The duration of the blank signal condition is determined according to a minimum blank time requirement, or extended by the metered approach traffic if detectors are used on that approach:

(i) If the metered approach has detectors, *minimum blank time* and *maximum blank extension time settings* are employed with a *gap setting* for extending the blank time. The timer for the maximum blank extension time setting starts after the minimum blank time (if a queue detector demand is registered on the controlling approach). The demand for the blank phase is registered when the first vehicle (during the red phase) is detected by the metered approach detectors.

(ii) If the metered approach does not have detectors, only a *minimum blank time setting* is used. The blank period may be terminated after the minimum blank time as soon as a queue detector demand is registered on the controlling approach. The demand for the blank phase is registered automatically as soon as the red phase is introduced.

The introduction and duration of the red signal on the metered approach is determined by the controlling approach traffic. For this purpose, *minimum red time* and *maximum red extension time settings* are used with a *queue detector gap setting* for extending the red time. A *queue detector occupancy setting*, i.e. the occupancy time for queue detection, is used as an additional queue detector setting to register the demand for the red signal phase. The timer for the maximum red extension time setting starts after the minimum red time (when demand is registered on the metered approach as described above). *Table 1* summarizes design and control parameters used for metering signals at various roundabouts in Melbourne, Australia.

Table 1 - Typical design and control parameters used for roundabout metering signals

Metered approach	
Signal stop-line setback distance	14 -24 m (46 - 79 ft)
Detector setback distance (if detector is used)	2.5 m (8 ft)
Loop length (if detector is used)	4.5 m (15 ft)
Minimum blank time setting	20 - 50 s
Maximum blank extension time settings	30 s
Blank signal gap setting	3.5 s
Yellow time	4.0 s
All-red time	1.0 - 2.0 s
Controlling approach	
Queue detector setback distance	50 - 120 m (164 - 394 ft)
Loop length for the queue detector	4.5 m (15 ft)
Minimum red time setting	10 - 20 s
Maximum red extension time settings	20 - 60 s
Queue detector gap setting	3.0 - 3.5 s
Queue detector occupancy setting: t_{oq}	4.0 - 5.0 s
Yellow time: t_{yR}	3.0 - 4.0 s
All-red time: t_{arR}	1.0 - 2.0 s

CASE STUDY: MICKLEHAM RD AND BROADMEADOWS RD ROUNDABOUT, MELBOURNE, AUSTRALIA

A method is described in this section for the analysis of roundabout capacity and performance characteristics with metering signals, using a case study described in Chapter 12 of the Australian (AUSTROADS) Roundabout Guide *(3)*. This is the intersection of Mickleham Rd and Broadmeadows Rd in Melbourne, Australia, a large Y-shaped multi-lane roundabout shown in *Figures 7 and 8*.

The AUSTROADS Guide states that, as a result of unbalanced flow conditions at this roundabout, the heavy traffic movement from Mickleham Rd South to Broadmeadows Rd caused long delays to the other heavy traffic movement from Mickleham Rd North to South, with extensive queuing (500 m to 600 m) occurring regularly during the morning peak on the Mickleham Rd North approach *(3)*. The Guide explains that metering signals consisting of two-aspect (red and yellow) signal faces were installed on the South approach. The signals were actuated by a queue of vehicles extending back along the North approach to the presence detectors located 90 m (about 300 ft) upstream of the yield line. The metering signals were found to reduce the queuing on the North approach substantially.

Results of analysis of this roundabout without metering signals were published previously showing that aaSIDRA was able to estimate the congestion observed at this roundabout while various other methods, including the analysis method described in the AUSTROADS Guide and the UK (TRL) linear regression model, estimated satisfactory operating conditions *(11,12,14)*.

For the purpose of this paper, the case has been modified for driving on the right-hand side of the road, and the analysis has been carried out using US units. The roundabout geometry data are given in *Table 2*. Volume data have been modified to demonstrate a case of reasonably large benefit from the use of metering signals. The volumes used in this analysis are given in *Figure 8*.

Also for the purpose of this paper, a more balanced lane use arrangement has been specified for the Mickleham Rd South approach compared with the original case. As seen in *Figure 8*, an exclusive left-turn lane and a shared left-turn and through lane are specified representing more balanced distribution of lane flows on the South approach, therefore in front of the Mickleham Rd North approach. In the analysis, these lane arrangements are used for conditions with and without metering signals. The original case has a shared left-turn and through lane plus an exclusive through lane on the Mickleham Rd South approach as seen in *Figure 7*. This results in a defacto exclusive left-turn lane and therefore a very heavy circulating flow rate in a single lane in front of the Mickleham Rd North approach, which is significantly less favorable in terms of capacity and performance of this approach.

The Analysis Method

The following analysis method was applied for modeling the effects of metering signals, which involved estimating operating characteristics for three operation scenarios using aaSIDRA:

(i) **Base Conditions**, i.e. roundabout operating **with BLANK metering signals** on the metered approach (Mickleham Rd Northbound) as shown in *Figures 7 and 8* (i.e. vehicles from the metered approach can enter the roundabout when gaps are available in the circulating road traffic). This corresponds to normal operation of the roundabout without metering signals.

(ii) Roundabout operation when the **metering signals display RED**, i.e. the metered approach traffic is stopped and the rest of roundabout operates according to normal roundabout rules as shown in *Figure 9*.

(iii) Signalized intersection scenario to emulate the operation of **metered approach signals** in order to determine the performance of the metered approach. The phasing information with red and blank phases is shown in *Figure 10*.

Figure 7 - Mickleham Rd and Broadmeadows Rd Roundabout, Melbourne, Australia
(all photos and drawings in this paper are modified to show driving on the right-hand side of the road)

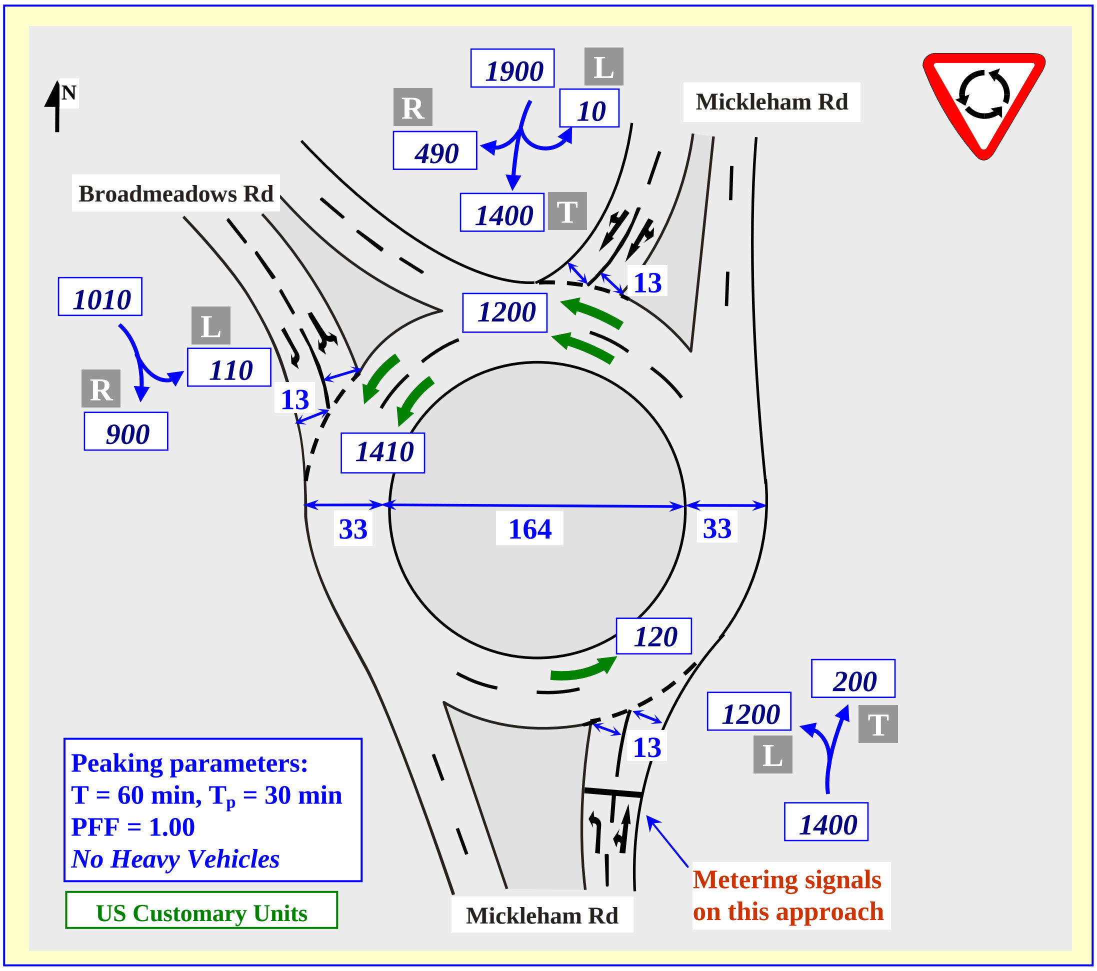

Figure 8 - Mickleham Rd and Broadmeadows Rd Roundabout, Melbourne, Australia

Table 2 - Geometry data for the Mickleham Rd and Broadmeadows Rd Roundabout

Approach ID	Approach Name	Average entry lane width	Total entry width	App. half width	Flare length (effective)	Entry radius	Entry angle (deg)
S	Mickleham Rd NB	13 ft (4.0 m)	26 ft (8.0 m)	23 ft (7.2 m)	66 ft (20.0 m)	319.4 ft (97.4 m)	40
N	Mickleham Rd SB	13 ft (4.0 m)	26 ft (8.0 m)	23 ft (7.2 m)	66 ft (20.0 m)	200.0 ft (60.0 m)	40
NW	Broadmeadows Rd	13 ft (4.0 m)	26 ft (8.0 m)	23 ft (7.2 m)	66 ft (20.0 m)	319.4 ft (97.4 m)	40
		Inscribed diameter	Central island diameter	Circulating road width	No of entry lanes	No of circulating lanes	
S	Mickleham Rd NB	230 ft (70.0 m)	164 ft (50.0 m)	33 ft (10.0 m)	2	2	
N	Mickleham Rd SB	230 ft (70.0 m)	164 ft (50.0 m)	33 ft (10.0 m)	2	2	
NW	Broadmeadows Rd	230 ft (70.0 m)	164 ft (50.0 m)	33 ft (10.0 m)	2	2	

The parameter values in metric and US customary units are not necessarily precise converted values.

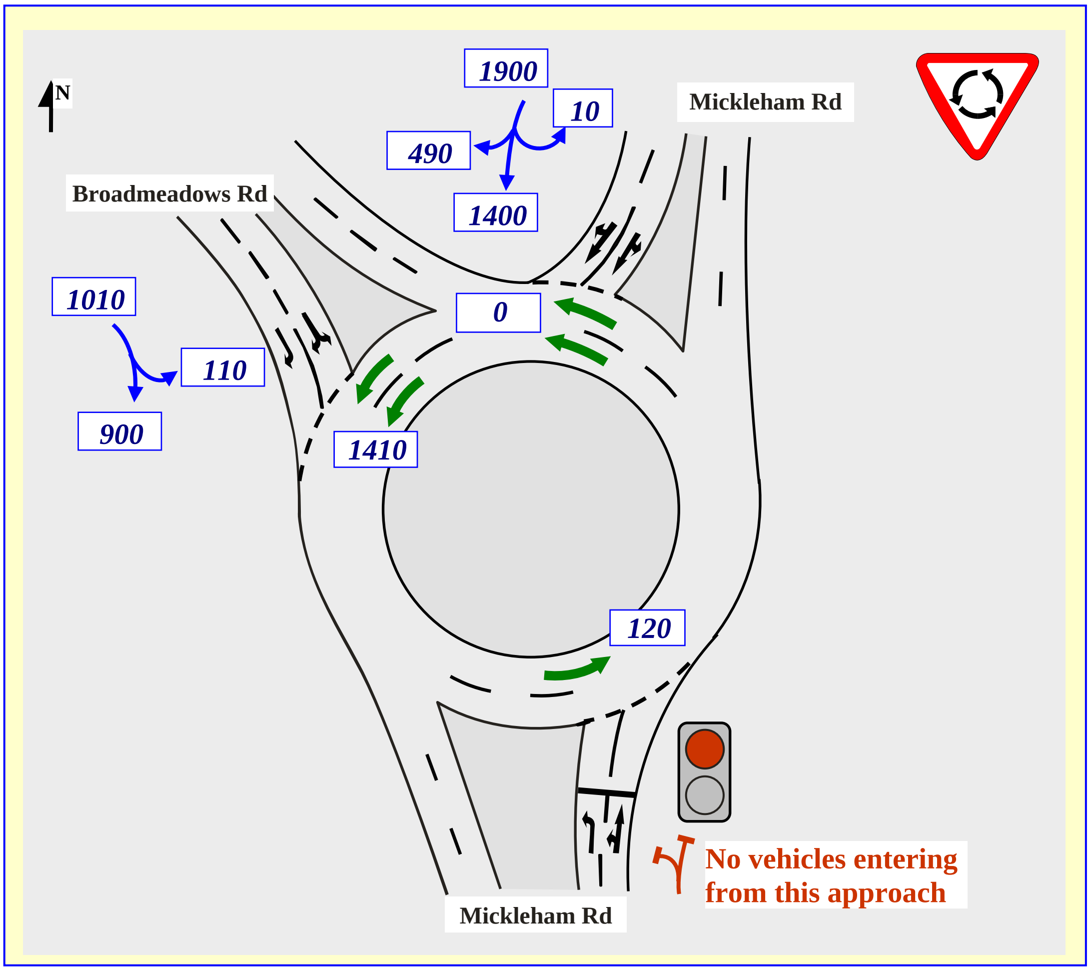

Figure 9 - Conditions of the Mickleham Rd and Broadmeadows Rd Roundabout with metering signals on Mickleham Rd South approach displaying red signal

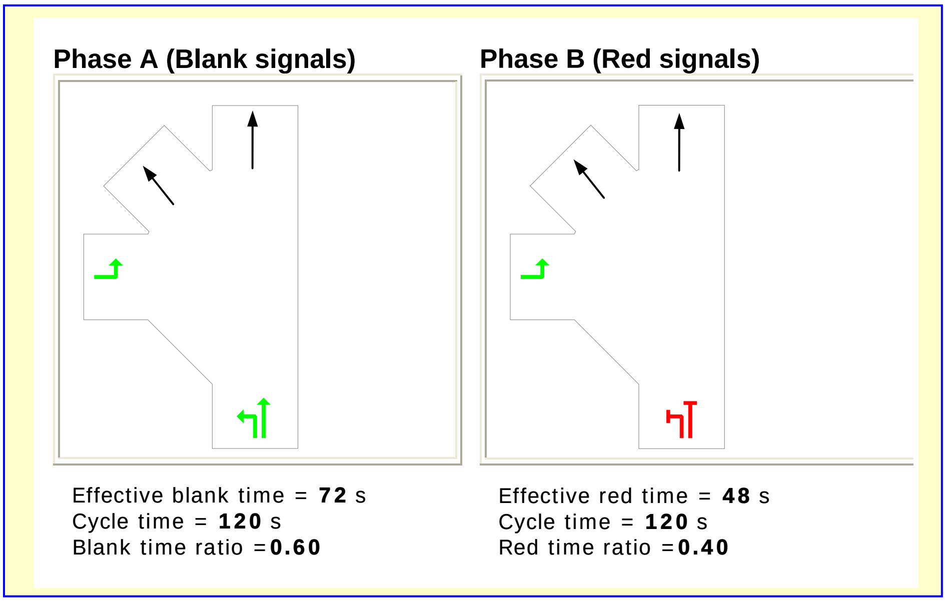

Figure 10 - Metering signal phasing for the Mickleham Rd and Broadmeadows Rd Roundabout

Standard default values of aaSIDRA 2.1 roundabout calibration parameters were used for this analysis.

In the signalized intersection scenario (iii), the saturation flow rate for each lane of the Mickleham Rd Northbound approach during the blank signal phase is specified as the capacity rate estimated for the case of normal roundabout operation as in the Base Conditions scenario (i). The same method is used for permitted (filter) turns and slip-lane movements in signalized intersection modeling. The saturation flow rates used were 1358 veh/h for Lane 1 (left-turn only) and 1584 veh/h for Lane 2 shared left-turn and through). This is probably a conservative assumption since it is likely that the saturation flow rate would be higher (shorter follow-up headway and critical gap values at the roundabout yield line) due to conditions of departure after queuing at upstream red signal.

For estimating geometric delay, operating cost, fuel consumption and emissions, it is important that the intersection negotiation data (turn radius, negotiation speed and negotiation distance) for the Mickleham Rd Northbound approach in the signalized intersection scenario (iii) are specified to match the data used in the Base Conditions scenario (i).

Using the signal timing information from scenario (iii), it was determined that the Mickleham Road Northbound approach traffic could be stopped for up to 40 per cent of the time without deteriorating its performance to unacceptable levels. This is based on the use of a 120 sec cycle time and allocating sufficient blank time to the metered approach considering acceptable conditions in terms of the degree of saturation (0.79), average delay (32 s), level of service (LOS C) and the longest 95th percentile back of queue in any lane (33 veh). In comparison, the performance statistics for this approach without metering signals are degree of saturation (0.48), average delay (12 s), level of service (LOS B) and the 95th percentile back of queue (5 veh). It is seen that the back of queue value for the metered approach is increased significantly due to the red signal effect.

To determine the overall impact of metering signals on the North and Northwest approaches, weighted average values of performance measures (capacity, delay, queue length, operating cost, etc) estimated by aaSIDRA for scenarios (i) and (ii) are calculated (60 per cent blank signal conditions and 40 per cent red signal conditions).

The capacity and performance results from the analysis are presented in *Table 3*. For some performance measures (CO_2, operating cost, etc), hourly values are converted to annual sums assuming 240 days per year (1 h per day) for the peak conditions. The annual sums as well as the benefits from the use of metering signals in this case are shown in *Figure 11*.

The analysis results indicate that:

(i) As expected, metering signals reduce delay and queue length on the controlling approach (Mickleham Road Southbound) but increase delay and queue length on the metered approach (Mickleham Road Northbound). This limits the overall benefit that can be obtained from metering signals.

(ii) For the controlling approach, the average delay and queue length appear to be reduced significantly (20 - 40 per cent). Availability of queue storage distance on the metered approach is a limiting factor that determines the proportion of time when red signal can be displayed, therefore limiting the benefits from metering signals.

(iii) Operating cost saving and CO_2 and other emission reductions are significant and may offer a good benefit-cost ratio due to the low cost of implementing metering signals.

Table 3 - aaSIDRA estimates of the performance of the Mickleham Rd and Broadmeadows Rd Roundabout with and without metering signals

Without metering signals (Base condition)								
App. ID	Approach Name	Dem Flow (veh/h)	Degree of saturation (v/c ratio)	Aver Delay (sec)	Level of Service	95% Back of Queue (veh)	CO2 (kg/h)	Oper. Cost ($/h)
S	Mickleham Rd NB	1400	0.48	12.1	**B**	4.7	373.4	363.52
N	Mickleham Rd SB	1900	1.06	82.8	**F**	61.7	708.8	904.14
NW	Broadmeadows Rd	1010	0.82	17.9	**B**	10.2	284.4	278.92
Intersection		**4310**	**1.06**	**44.6**	**D**	**61.7**	**1366.6**	**1546.58**
With metering signals (red signal 40 per cent of the time)								
App. ID	Approach Name	Dem Flow (veh/h)	Degree of saturation (v/c ratio)	Aver Delay (sec)	Level of Service	95% Back of Queue (veh)	CO2 (kg/h)	Oper. Cost ($/h)
S	Mickleham Rd NB	1400	0.79	31.7	**C**	33.1	408.80	437.92
N	Mickleham Rd SB	1900	0.77	52.4	**D**	37.0	612.24	720.94
NW	Broadmeadows Rd	1010	0.68	15.7	**B**	8.0	278.48	271.10
Intersection		**4310**	**0.79**	**37.1**	**D**	**37.0**	**1299.52**	**1429.96**

Mickleham Rd SB (Southbound): Controlling approach

Mickleham Rd NB (Northbound): Metered approach

Intersection level of service given above is based on the average intersection delay (not the worst movement delay).

Option	Description	Demand Flow	Average Delay	Worst Approach Delay	95% Back of Queue	Total Stops	CO2	Operating Cost
		(veh/h)	(sec)	(sec/veh)	(veh)	(veh/y)	(kg/y)	($/y)
1	No Metering Signals	1,034,400	44.6	82.8	62	1,870,248	327,984	371,179
2	With Metering Signals	1,034,400	37.1	52.4	37	1,479,019	311,885	343,190
	Difference	0	-8	-30	-25	-391,229	-16,099	-27,989
	Per cent difference	0.0%	-16.9%	-36.7%	-40.0%	-20.9%	-4.9%	-7.5%

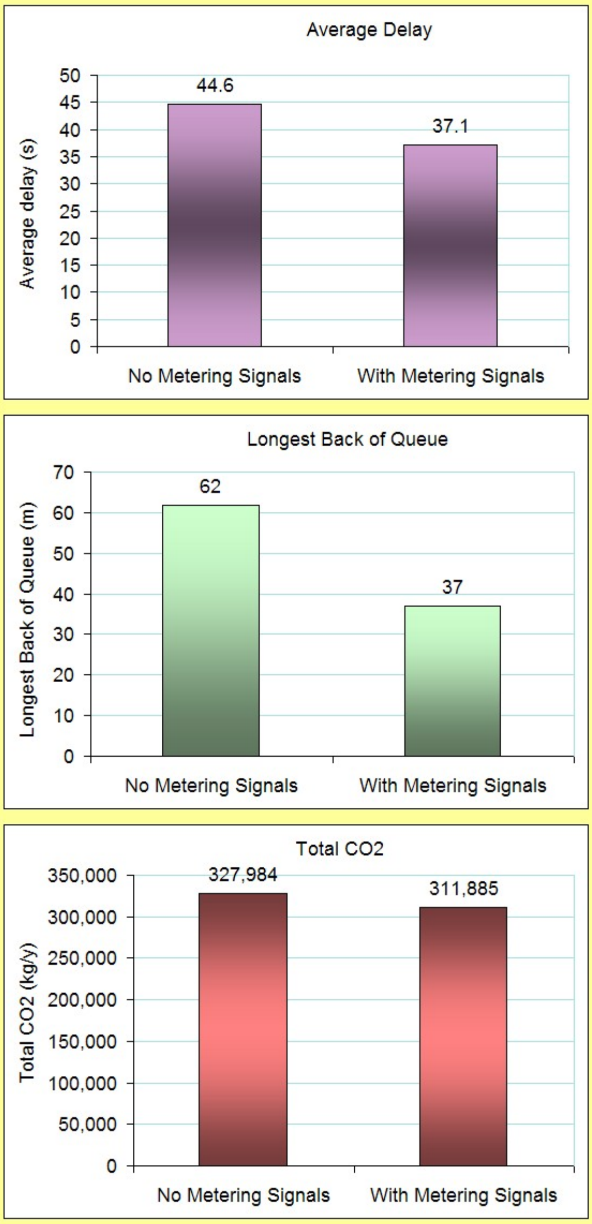

Figure 11 - Benefits from metering signals for the Mickleham Rd and Broadmeadows Rd Roundabout

CONCLUDING REMARKS

The analysis method described in this paper is an approximate one which involves various assumptions. It is possible that benefits from the metering signals are higher than indicated in this paper considering dynamic variations in demand flows in real-life traffic conditions and residual effects of oversaturated conditions continuing after the periods analyzed. A more comprehensive method has been developed and will be included in a future version of aaSIDRA.

Field observations are recommended on driver behavior at roundabouts subject to metering signal control. In particular, the "saturation flow rate" of the metered approach should be compared with the "capacity rate" of normal roundabout operation (without metering signals) to establish if the metering signals affect driver behavior, i.e. if those vehicles queued at a red signal subsequently display shorter queue discharge (follow-up) headways and accept shorter gaps.

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