Roundabouts with Unbalanced Flow Patterns

Rahmi Akçelik

Akcelik and Associates Pty Ltd
PO Box 1075G, Greythorn Victoria 3104, Australia
Phone: +613 9857 9351
Fax: +613 9857 5397
Email: rahmi@akcelik.com.au

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- **Right-hand version figures with US customary units and additional material included for some case studies (see the Appendix after references).**
- **Some data tables extended to include data in both metric and US customary units.**
- **Critical lanes in Table 5 are better identified, and Equation 4 corrected.**

Revised: 6 May 2005

- **Moore St and Fitzsimons Lane roundabout cases revised and US case results added in the Appendix (U-turns shown in figures for Moore St roundabout).**

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ABSTRACT

Complex interactions among the geometry, driver behavior, traffic stream and control factors determine the roundabout capacity and level of service. With increased use of modern roundabouts, an improved understanding of the effect of origin-destination demand pattern of traffic on roundabout capacity and level of service will help 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. Unbalanced flows may not be a problem when the overall demand level is low but appear with traffic growth even at medium demand levels. Modeling of traffic demand pattern is important in optimizing the roundabout geometry including approach and circulating lane use. Case studies are presented to show that roundabout capacity and level of service depend not only on the circulating flow rate but also the characteristics of approach flows contributing to the circulating flow. The amount of queuing on the approach road, circulating lane use, priority sharing and priority emphasis are the factors that need to be taken into account. Dominant circulating flows that originate mostly from a single approach with high levels of queuing and unequal lane use (with most vehicles in one lane), cause priority emphasis and reduce the entry capacity significantly. This is evident from the use of part-time metering signals under peak demand conditions in order to alleviate the problem of excessive delay and queuing by creating gaps in the circulating stream. This is a cost-effective measure to avoid the need for a fully signalized intersection treatment. The Australian roundabout and traffic signal guides acknowledge the problem and discuss the use of metering signals.

Figure 1 - An example of dominant entry flow at a modern roundabout in Australia
(photo modified for driving on the right-hand side of the road)

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, entry angle, 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.

Current discussion on roundabout models appears to concentrate on capacity alone without much discussion of performance (delay, queue length, emissions, etc). A simplistic view of roundabout capacity models considers analytical models only, and classifies them into two mutually exclusive categories, namely "theoretical (gap-acceptance) only" and "empirical only". This view presents the US Highway Capacity Manual and Australian (aaSIDRA, AUSTROADS, NAASRA) gap-acceptance based models *(1-12)* as belonging to the first group and a linear regression model developed by TRL (UK) *(13-19)* as belonging to the second group. As the use of roundabouts became more common in the USA, this narrow view resulted in some controversy as competing software packages based on the two categories, namely aaSIDRA representing the gap-acceptance methodology and the ARCADY and RODEL representing the TRL linear regression model, presented significant differences in some cases *(10,20-22)*. The issue, while narrowly focused, has been discussed widely among traffic engineering professionals in the USA *(23, 24)*, and has already been a subject of debate among researchers and practitioners *(25-30)*. The author has presented various thoughts about the limitations of the Australian gap-acceptance models as well as the UK linear regression model previously *(10,22)*.

Table 1 - Summary of field data at 55 roundabout lanes used for calibrating the Australian gap-acceptance based capacity and performance models (points not used in critical gap and follow-up headway regressions not included)

	Total entry width (m)	No. of entry lanes	Average entry lane width (m)	Circul. width (m)	Inscribed Diameter (m)	Entry radius (m)	Conflict angle (°)
Minimum	3.7	1	3.20	6.5	16	4	0
Maximum	12.5	3	5.50	12.0	220	∞	80
Average	8.1	2	3.84	9.6	56	39	29
15th percentile	6.4	2	3.34	8.0	28	10	0
85th percentile	10.5	3	4.48	11.9	70	39.8	50
Count	55	55	55	55	55	55	55

	Follow-up Headway (s)	Critical Gap (s)	Crit. Gap / Fol. Hw Ratio	Circul. flow (veh/h)	Total entry flow (veh/h)	Dominant lane flow (veh/h)	Subdom. lane flow (veh/h)
Minimum	0.80	1.90	1.09	225	369	274	73
Maximum	3.55	7.40	3.46	2648	3342	2131	1211
Average	2.04	3.45	1.75	1066	1284	796	501
15th percentile	1.32	2.53	1.26	446	690	467	224
85th percentile	2.65	4.51	2.31	1903	1794	1002	732
Count	55	55	55	55	55	55	55

In fact, the difference is between a *linear regression* model and a *gap-acceptance based* model not between an *empirical* and a *theoretical* model. The current Australian and US HCM models based on gap-acceptance modeling do have an empirical base *(4,6)*. The *Australian* gap-acceptance model *(7,8,10,11)* uses gap-acceptance parameters calibrated by field surveys conducted at a large number of modern roundabouts in Australia *(6)*. *Table 1* shows a summary of field data at 55 roundabout lanes used for calibrating the Australian gap-acceptance based capacity and performance models.

There are significant differences between various gap-acceptance models, e.g. a model that uses fixed gap-acceptance parameters *(1,5)* and another model that determines gap-acceptance parameters as a function of roundabout geometry and traffic flow conditions using empirical relationships *(6-8,12)*. Similarly, there is no reason why a linear regression model could not be based on a lane-by-lane *(12,29,30)* or lane group *(1)* approach and include other parameters related to driver behavior rather than treating all traffic using the approach road as a whole and being limited to roundabout geometry parameters only *(13-19,29,30)*. Thus, it is necessary to investigate the available models in a general framework, considering all aspects of models relevant to roundabout operation. Microsimulation models should be included in this general framework of discussion since most modeling issues relevant to analytical models are relevant to microsimulation models as well *(31)*.

This paper discusses an important factor that influences the capacity and performance of entry stream, namely the origin-destination pattern of arrival (demand) flows as related to the approach and circulating lane use (see *Figure 1*). This impacts headway distributions of circulating streams, and as a result, affects approach lane capacities and performance. This factor is not taken into account in the TRL (UK) linear regression model, the HCM and the Australian gap-acceptance models other than aaSIDRA, or any other regression or gap-acceptance models known to the author. The related issues of *priority reversal* and *priority emphasis* are also discussed briefly.

For the practitioner, it is important to understand the reasons behind systematic differences between estimates from different models so that judgment can be made about accepting or rejecting results of a particular model, or a given model can be calibrated, in a specific situation.

Four case studies of unbalanced flows reported by the author in previous publications are summarized in the following section (one-lane, two-lane and three-lane roundabouts from Australia and the USA with total intersection flow rates in the range approximately 2300 to 5300 veh/h). Three new case studies comparing capacity and performance estimates from different models are presented in this paper after discussing the unbalanced flow issue (one-lane, two-lane and three-lane roundabouts from Australia and UK with total intersection flow rates in the range approximately 1700 to 5000 veh/h).. These case studies also demonstrate the importance of modeling different approach and circulating lane arrangements at multi-lane roundabouts.

PREVIOUS CASE STUDIES

The following case studies comparing alternative models for single-lane and multi-lane roundabouts, demonstrating the importance of allowing for the origin-destination pattern of arrival (demand) flows, have been reported by the author previously.

Intersection of Mickleham Road and Broadmeadows Road in Melbourne, Australia

This is a large Y-shaped multi-lane roundabout with an inscribed diameter of 60 m (see *Figure 2*) described in Chapter 12 of the Australian Roundabout Guide *(8)*. The capacity of the North approach (Mickleham Road) is very low due to heavy right-turn flow from the South approach (1400 veh/h single lane circulating flow), and "extensive queuing (500 m to 600 m) occurred regularly during the morning peak" *(8)*. Results of analysis of this roundabout were published previously showing that aaSIDRA was able to estimate the congestion observed at this roundabout but the UK (TRL) linear regression model *(13-19)* and the method described in the Australian Roundabout Guide *(8)* estimated satisfactory operation *(10,20,21)*. To improve the conditions for traffic on the North approach, part-time metering signals are used. These are installed on the South approach of the roundabout, and actuated by the queue of vehicles extending back along the North approach onto detectors 90 m upstream of the give-way line. aaSIDRA estimates that allocation of two lanes to the right-turn movement from South would alleviate this problem.

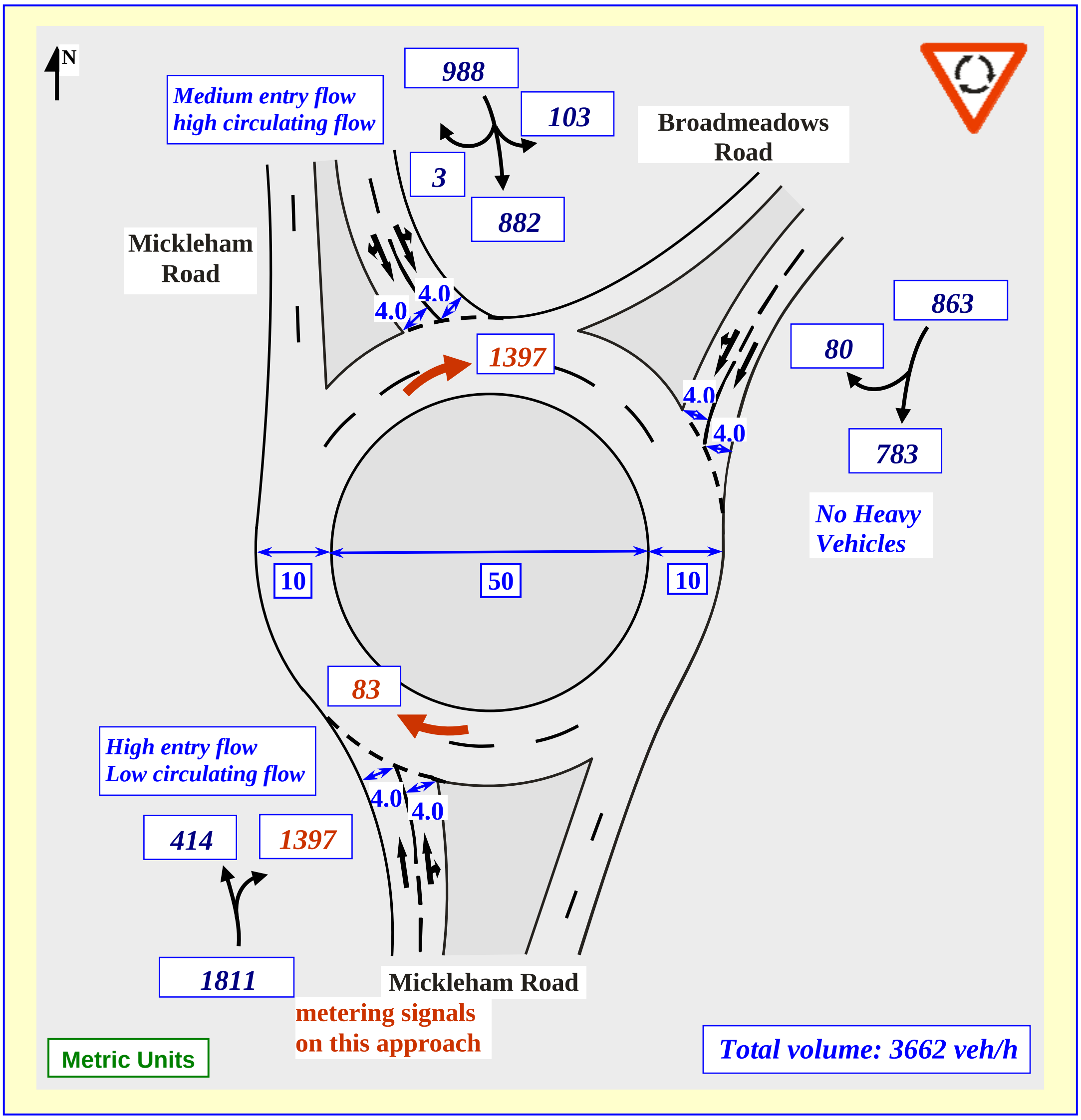

Figure 2 - Intersection of Mickleham Road and Broadmeadows Road in Melbourne, Australia: an unbalanced flow case reported previously (10,20,21).
Driving on the left-hand side of the road applies (see the Appendix for driving on the right-hand side of the road with US customary units).

Intersection of Parkes Way, Kings Avenue and Moreshead Drive in Canberra, Australia

This is a large 4-leg multi-lane roundabout in Canberra, Australia with an inscribed diameter of 145 m (see *Figure 3*). The analysis of conditions before and after changing lane arrangements on one approach road was conducted. Queues up to 3 km long and excessive delays were observed on Moreshead Drive where lane disciplines were changed from L, T, TR to L, T, R. This case presented a problem of unbalanced flow caused by the heavy right-turn flow from Parkes Way.

This movement was observed to operate at capacity, which is estimated accurately by the aaSIDRA method. This dominant flow reduces the capacity of the Moreshead Drive approach, causing extensive queuing and long delays in the through lane (single lane) in the After case. The method described in the Australian Roundabout Guide *(8)* and the UK (TRL) linear regression method failed to estimate the congestion that results from this change to lane arrangements. The UK (TRL) model is not sensitive to different lane arrangements *(29,30)*, and therefore gives the same results for both before and after conditions. aaSIDRA estimates are found to be satisfactory due to the method that allows for unbalanced flow conditions.

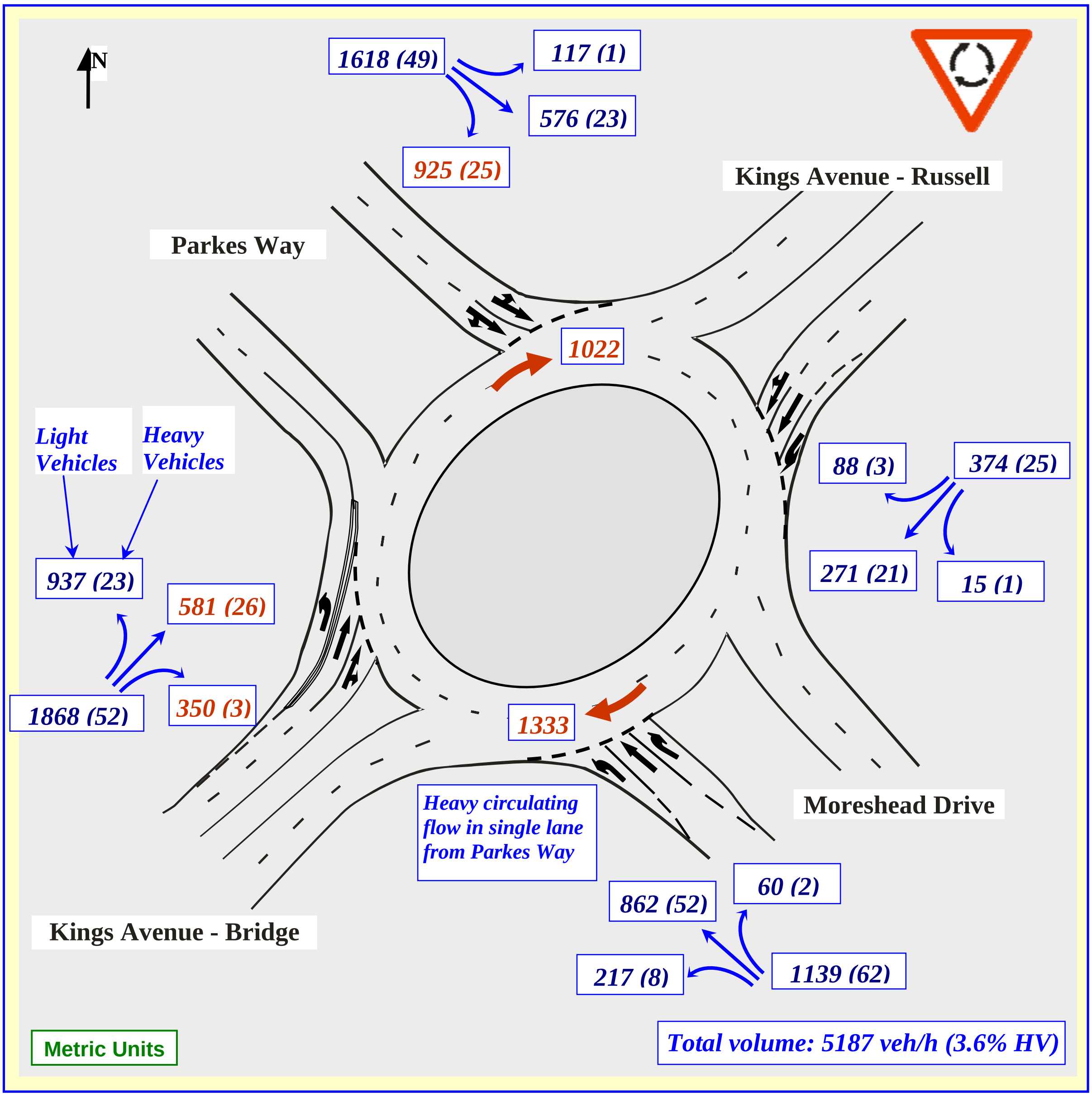

Figure 3 - Intersection of Parkes Way, Kings Avenue and Moreshead Drive in Canberra, Australia: an unbalanced flow case reported previously *(10,20,21)*
Driving on the left-hand side of the road applies.

Intersection of Fitzsimons Lane and Porter Street in Melbourne, Australia

A highly congested two-lane roundabout was redesigned as a three-lane roundabout eliminating persistent congestion (see *Figure 4*). Detailed field surveys were carried out before and after the reconstruction *(32)*. Metering signals exist on the North approach (Fitzsimons Lane) to help the Porter Street traffic. aaSIDRA provided satisfactory capacity and performance estimates for before and after conditions at this roundabout while the other methods failed to provide satisfactory estimates especially for highly saturated before conditions.

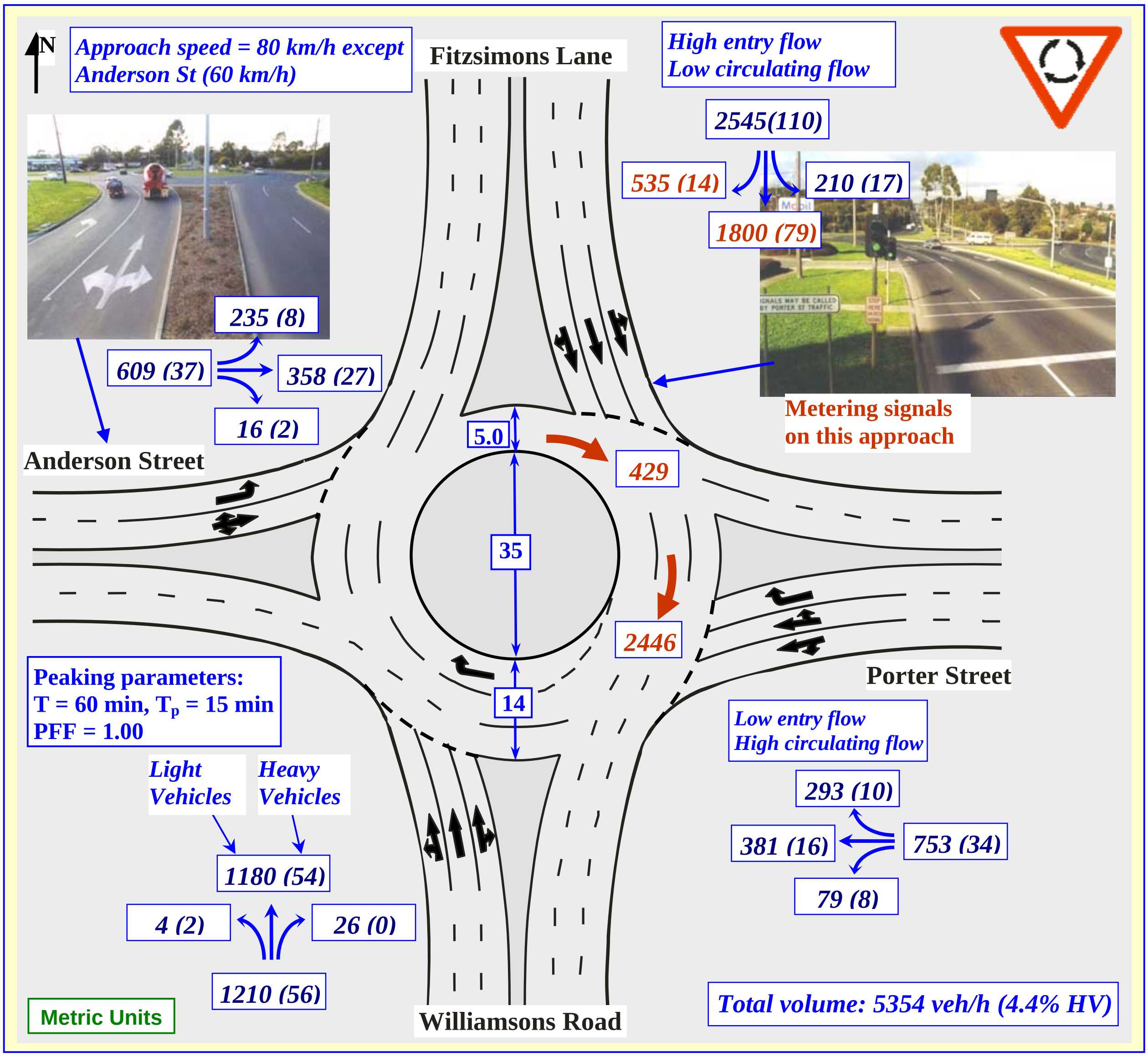

Figure 4 - Intersection of Fitzsimons Lane and Porter Street in Melbourne, Australia: an unbalanced flow case reported previously *(32)*
Driving on the left-hand side of the road applies (see the Appendix for driving on the right-hand side of the road with US customary units).

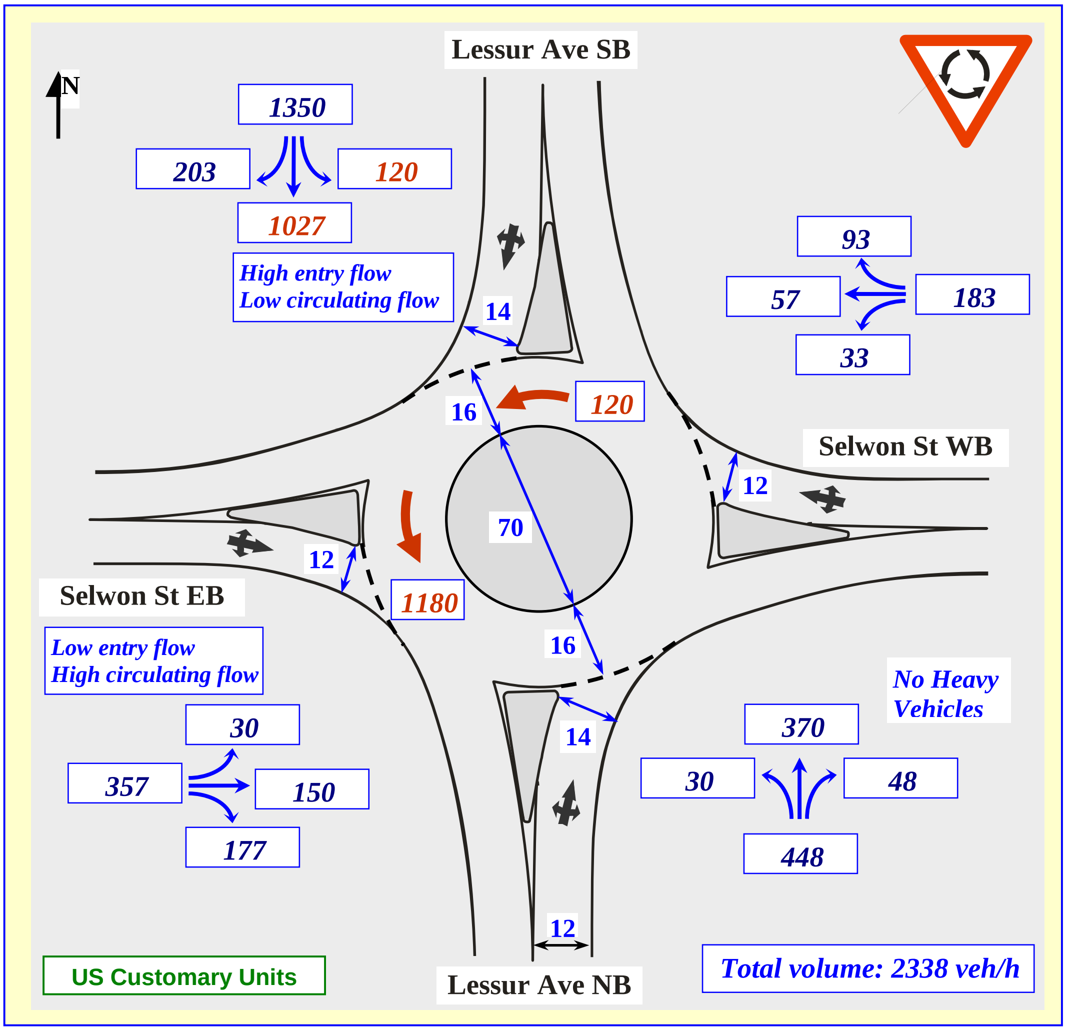

Figure 5 - A single-lane roundabout, USA: an unbalanced flow case reported previously *(22)*

A US Roundabout Case

(geometric data and results given in the appendix)

A small-size single-lane roundabout based on a case from a US city is analyzed (see *Figure 5*). The road names are modified due to confidentiality reasons. This case is discussed in detail in a recent paper by the author *(22)*. This roundabout has heavy North - South through movement volumes on Lessur Ave, and low volumes on East and West approaches (Selwon St). This situation may arise when a roundabout is considered as an alternative treatment to replace two-way stop control at a major road intersection where low minor road volumes are opposed by high major road volumes.

As in other unbalanced flow cases, this case presents a combination of (i) high approach flow - low circulating flow and (ii) low approach flow, high circulating flow as a result of highly directional (unbalanced) flows (see *Figure 5*). This contributes to significant differences in capacity estimates for Southbound and Eastbound approaches from the aaSIDRA and other capacity models. Since this is a single-lane roundabout case, the HCM 2000 model is also considered. While aaSIDRA indicates sufficient capacity for the Southbound approach (high approach flow, low circulating flow), it estimates oversaturated conditions for the Eastbound approach (low approach flow, high circulating flow). The UK (TRL) Linear Regression and the HCM 2000 models estimate opposite conditions compared with aaSIDRA since these models do not take account of such unbalanced flow cases.

UNBALANCED FLOWS AT ROUNDABOUTS - THE ISSUE

Improved understanding of the effect of 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 *(10,20-22,32)*. 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 *(10,22,32,33)*, UK *(34-39)* and USA *(40)* to alleviate the problem of excessive delay and queuing by creating gaps in the circulating stream.

Huddart's *(34)* 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.*"

Unbalanced flow conditions may arise at T-intersection and freeway interchange roundabouts as well as normal four-way intersections as seen in the case studies given in this paper.

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.

- Small to medium size single-lane roundabout at the intersection of Grange Rd, St Georges Rd and Alexandra Avenue in Toorak (see *Figure 1*): **1693 veh/h per lane** entering against a circulating flow rate of 67 veh/h has been reported (6). 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.
- Large multi-lane roundabout at the intersection of Mickleham Rd and Broadmeadows Rd in Westmeadows (see *Figure 2*): **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 (this case is described in the Australian Roundabout Guide (8) as discussed above). The sum of entering and circulating flows is 1480 veh/h in am peak and 1613 in pm peak.

- Small single-lane roundabout at the intersection of Stanhope Grove with Broadway in Camberwell (see *Figure 6*): **1524 veh/h per lane** entering against a circulating flow rate of 60 veh/h has been reported *(41)*. The sum of entering and circulating flows is 1584 veh/h.

Several studies related to the issue of unbalanced flows at roundabouts have been reported in the literature *(42,43)*. A recent study of a roundabout in Denmark *(43)* 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 6 - Stanhope Grove with Broadway Roundabout (Camberwell), Melbourne, Australia *(41).*
Driving on the left-hand side of the road applies.

Modeling of 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 *(32)*. 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 *(10,22,32,33)*. The signalized roundabout solution has been used extensively in the UK as well *(34-39)*. A recent US paper discusses the use of metering signals for the Clearwater roundabout in Florida *(40)*. The Australian roundabout and traffic signal guides acknowledge the problem and discuss the use of metering signals *(8,33)*.

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 7 shows the use of roundabout metering signals and an example from Melbourne, Australia (picture modified to show driving on the right-hand side of the road). The signalized approach is referred to as the *metered approach*, and the approach with the queue detector as the *controlling approach*. Two-aspect yellow and red signals are used. The sequence of aspect display is Off to Yellow to Red to Off. 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 midblock signalized crossing on the metered approach.

The Australian Traffic Signal Guide *(33)* 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 from the give-way (yield) line. Queue detector setback distance on the controlling approach is usually in the range 50 m to 120 m. In some cases, it may be necessary to supplement the traffic signals with explanatory fixed or variable message signposting. Where sight restrictions exist, advance warning signals are considered.

When the queue on the controlling approach extends back to the queue detector, the signals on the metered approach operate 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. The introduction and duration of the red signal on the metered approach is determined by the controlling approach traffic. The duration of the blank signal is determined according to a minimum blank time requirement, or extended by the metered approach traffic if detectors are used on that approach.

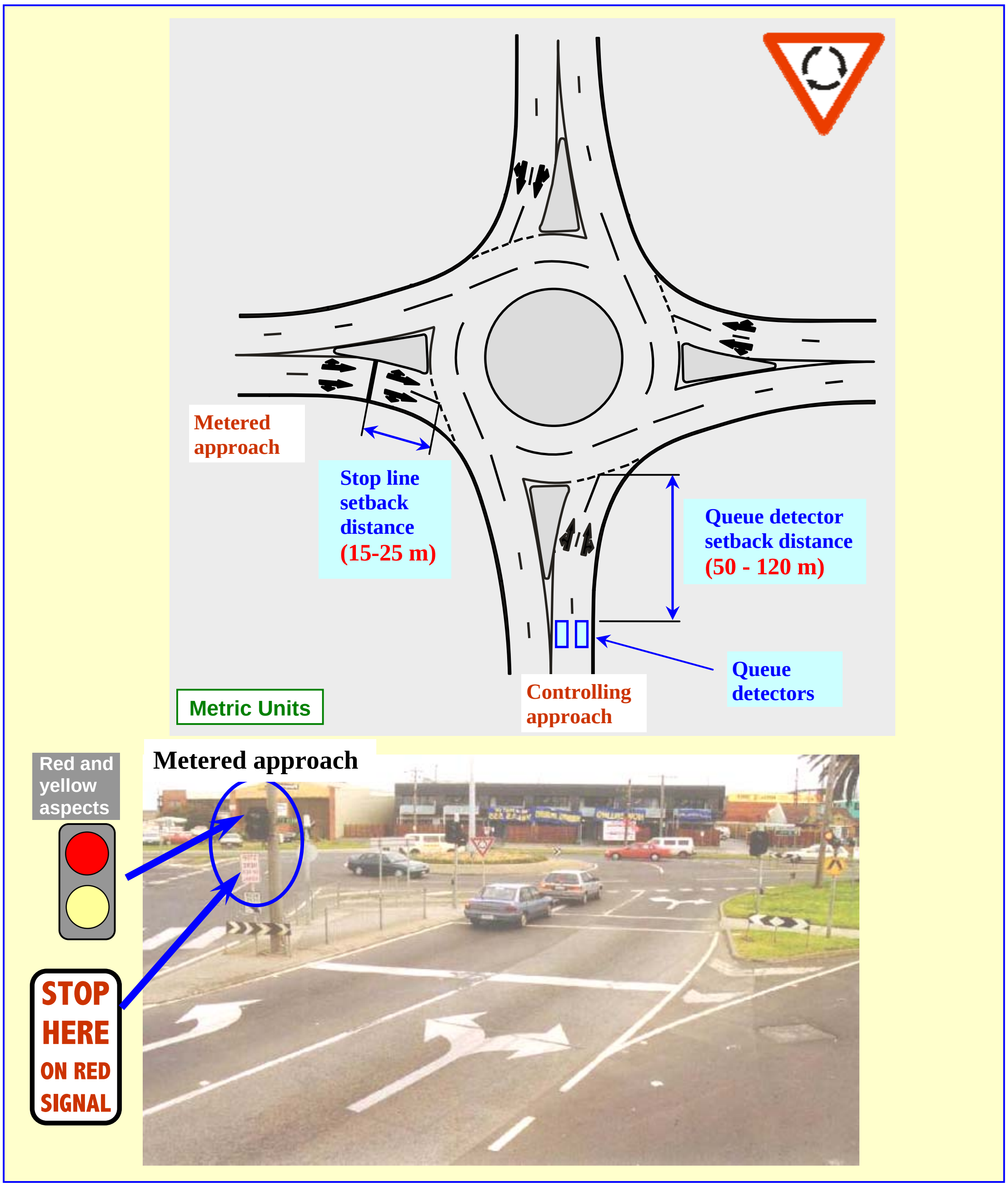

Figure 7 - Use of metering signals with an example from Melbourne, Australia (picture modified to show driving on the right-hand side of the road)

UNBALANCED FLOWS AT ROUNDABOUTS - THE aaSIDRA MODEL

All roundabout capacity models predict decreased capacity with increased circulating flow. In gap-acceptance modeling, this is due to the *blocked periods* that result when the approach vehicles cannot find an acceptable gap in the circulating stream. *Unblocked periods* represent when queued or unqueued vehicles can enter the circulating road when a gap is available in the circulating flow. Blocked and unblocked periods are like *effective red and green times* at signals, and the sum of blocked and unblocked times can be called the *gap-acceptance cycle time (9)*. Thus, roundabout gap-acceptance capacity can be expressed in the same way as capacity at traffic signals *(9,12)*:

$$Q_e = s\,u \qquad (1)$$
$$s = 3600 / \beta \qquad (2)$$
$$u = g / c \qquad \textit{subject to } u \geq u_{min} \qquad (3)$$

where

Q_e = entry lane capacity, i.e. the maximum flow rate that can enter the roundabout under prevailing conditions (veh/h),

s = saturation flow rate, i.e. the queue discharge flow rate when acceptable gaps occur (veh/h),

u = *unblocked time ratio* (ratio of the average unblocked time to the average gap-acceptance cycle time),

u_{min} = minimum unblocked time ratio corresponding to a minimum capacity value,

β = follow-up headway (saturation headway) (seconds),

g = average unblocked time (seconds), and

c = average gap-acceptance cycle time (seconds).

Note that the proviso "under prevailing conditions" is important in defining the capacity as a *service rate* rather than the maximum amount of traffic that the roundabout can carry since capacity at a roundabout drops significantly with increasing demand (therefore circulating) flows.

Many different forms of the roundabout capacity formula based on gap-acceptance method that exist, including the HCM capacity formula, can be explained in terms of the concept expressed by *Equation (1)*. aaSIDRA uses this concept directly to calculate the gap-acceptance capacity. In aaSIDRA version 2.1, the gap-acceptance capacity, Q_e incorporates the following effects:

(i) critical gap and follow-up headway of the entry stream depend on the roundabout geometry (inscribed diameter, number of entry lanes, average entry lane width and number of circulating lanes), the type of lane (dominant or subdominant) as well as the circulating flow and arrival (demand) flow rates; an *environment factor* for local conditions;

(ii) at low circulating flow rates, critical gap and follow-up headway decrease with increasing *ratio of demand flow rate to circulating flow rate* (a calibration factor is available for determining an appropriate level of the effect of this factor);

(iii) heavy vehicles in the circulating stream increase the effective circulating flow rate;

(iv) heavy vehicles in the entry stream increase the follow-up headway and critical gap values (decrease capacity);

(v) a bunched exponential distribution of circulating stream headways is used together with the critical gap parameter of the entry stream to determine the average unblocked time, average gap-acceptance cycle time and the unblocked time ratio;

(vi) minimum intrabunch headway, proportion bunched (or free) in the circulating stream and an *O-D factor* are the parameters that affect the distribution of circulating stream headways, therefore the unblocked time ratio;

(vii) effective number of circulating lanes based on the flow pattern in circulating lanes in front of each approach determines the values of intrabunch headway, proportion bunched and the *O-D factor;*

(viii) the proportion bunched (or free) varies with the circulating flow rate, and depends on the minimum intrabunch headway (therefore on the effective use of the circulating lanes); see *Equation (4)* below;

(ix) the *O-D factor* (f_{od}) is determined according to the origin-destination flow pattern (establishing dominant flow component of the circulating stream), proportioned queued in the approach lane used by each dominant stream component of the circulating stream, and the circulating lane use of all components of the circulating stream (as affected by the approach lane use); this factor also allows for the effect of any priority sharing between the entry and circulating streams (see discussion below);

(x) the critical gap, follow-up headway, average unblocked time, average gap-acceptance cycle time and the unblocked time ratio parameters are used not only in the capacity formula but also in all performance equations (delay, queue length, number of stops, and so on).

Proportion Bunched

The proportion bunched (or free) in the circulating stream is determined from the following formula (this replaces the exponential model used in earlier versions of aaSIDRA):

$$\varphi = (1 - \Delta\, q_c) / [1 - (1 - k_d)\, \Delta\, q_c] \qquad \textit{subject to } \varphi \geq 0.001 \qquad (4)$$

where

φ = proportion unbunched (free) in the circulating stream,

Δ = minimum intrabunch headway in the circulating stream (seconds),

q_c = circulating flow rate including the effect of heavy vehicles in the circulating stream (pcu/h),

k_d = bunching delay parameter (a constant), k_d = 2.2 for roundabout circulating streams.

The model given by *Equation (4)* was developed by considering a fundamental relationship between travel delay parameter in Akçelik's speed-flow function (44) and the bunching delay obtained through the bunching model to determine vehicle headway distributions. *Figure 8* shows the proportion unbunched for single-lane circulating streams at roundabouts (measured and estimated by alternative models). *Figure 9* shows the proportion unbunched for one-lane, two-lane and three-lane roundabouts using *Equation (4)* together with field data (6) for single-lane and multi-lane roundabouts.

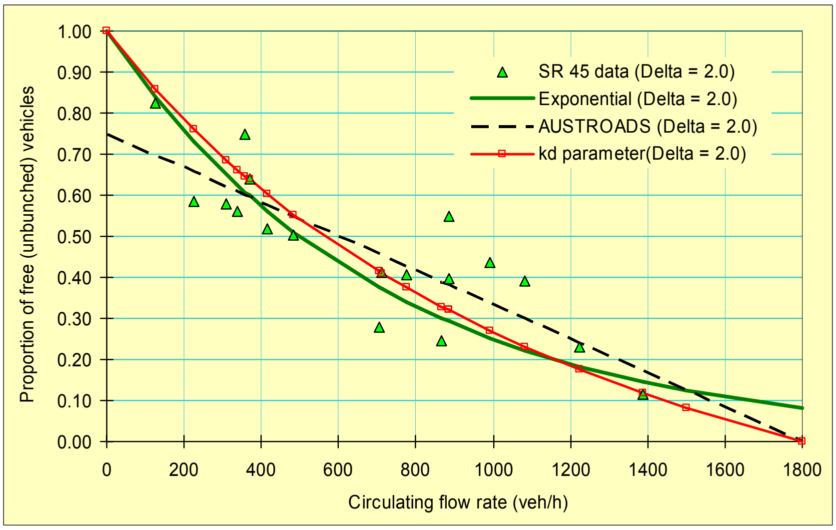

Figure 8 - Proportion unbunched for single-lane circulating streams at roundabouts as a function of the circulating flow rate (measured and estimated by alternative bunching models) (6,8,45)

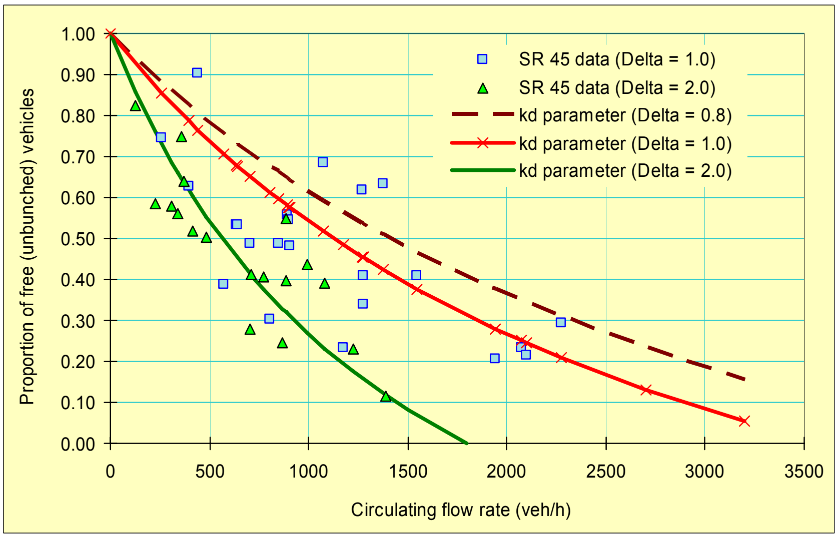

Figure 9 - Proportion unbunched for one-lane (Δ = 2.0 s), two-lane (Δ = 1.0 s) and three-lane (Δ = 0.8 s) roundabouts using the bunching model based on travel delay parameter

The Origin-Destination (O-D) Factor

The O-D factor was first introduced in an earlier version SIDRA to allow for unbalanced flow effects after research was conducted *(10,20,21,46-48)* following reports received from many practitioners that overoptimistic results were obtained using the Australian Roundabout Guide method *(8)* which did not allow for unbalanced flow effects. 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).

The O-D factor method represents a substantial change to the method described in the Australian Roundabout Guide from which aaSIDRA originated (8). The application of this factor to the unblocked time ratio in aaSIDRA 2.1 introduces another significant change to the model due to the direct use of this parameter in performance equations. *Figure 10* shows the effective unblocked time ratio as a function of circulating flow for three levels of O-D flow pattern effect (and without the O-D factor) for the dominant lane of a two-lane roundabout (inscribed diameter = 50 m, average lane width = 4.0 m, environment factor f_e = 1.0, medium adjustment level for the ratio of entry flow to circulating flow, entry flow rate = 900 veh/h).

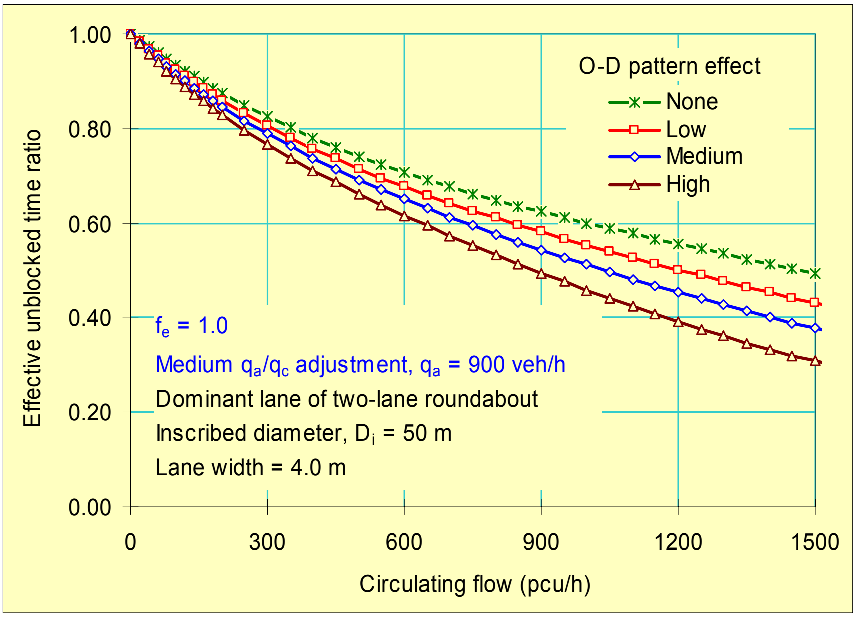

Figure 10 - Effective unblocked time ratio as a function of circulating flow for three levels of O-D flow pattern effect (and without the O-D pattern factor) for the dominant lane of a two-lane roundabout

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, which has been a concern expressed by many practitioners. The case studies reported previously and those presented in the next section are examples of such cases. In all real-life cases considered, the methods without the unbalanced flow modeling predict good operating conditions whereas long delays and queues are observed on one or more approaches of such roundabouts.

Figure 11 explains the effect of the O-D factor in aaSIDRA. 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.

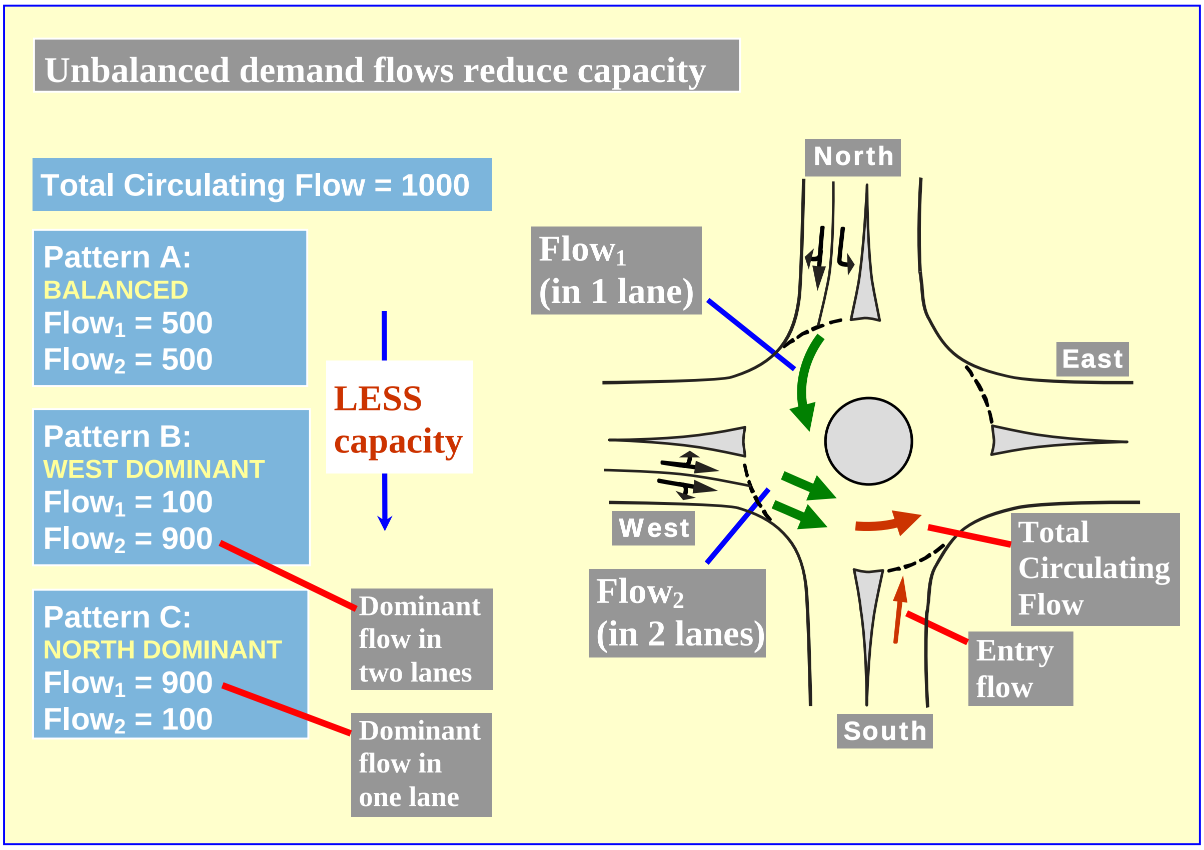

Figure 11 - The effect of the Origin-Destination (O-D) pattern on capacity in modeling unbalanced flows

Generally, the extent of the unbalanced flow problem is likely to be underestimated by the TRL (UK) linear regression model, HCM 2000 and the Australian 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. 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 *(49-51)* 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 $\beta + \Delta > \alpha$, where β = follow-up headway, α = critical gap (headway) and Δ = intra-bunch 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 Australian Roundabout Guide model *(8)*.

The O-D factor in aaSIDRA allows for the fact that vehicles entering from the approach queues are under *forced flow* conditions, and as such they are considered to be *bunched*. Without the O-D factor that reduces the unblock time ratio (in effect, modifying 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. 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.

Of particular concern is the application of the bunched exponential model of headway distribution to roundabout circulating streams without due attention to the headways of vehicles entering from approach queues, i.e. entering with follow-up (saturation) headway. Roundabout circulating streams are *uninterrupted* flows in short road segments on the circulating road (between *entry - circulating road junctions*), and they contain *queued vehicles* entering from approach lanes. Vehicles departing from a queue with follow-up headways are in forced flow conditions, and should be considered to be bunched when negotiating the roundabout even though the follow-up headway is longer than the intrabunch headway used in the general bunching model which is based on average circulating flow conditions (*Equation 4*).

Especially under heavy demand conditions, the proportion of queued vehicles in the circulating stream increases. Consideration of all headways above the intrabunch (capacity) headway as unbunched headways (although these are between vehicles entering from upstream approach queues with follow-up headways) can cause overestimation of capacity at the downstream entry.

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 (with follow-up headways that can be larger than the intrabunch headway), 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.

FURTHER CASE STUDIES

In addition to the four case studies published previously *(8,10,20-22,32)*, and summarized at the start of this paper, three more case studies are presented in this paper:

(i) a small single-lane freeway interchange roundabout in Sydney, Australia;

(ii) a two-lane T-intersection roundabout case based on an example published in the UK *(29,30)*; and

(iii) a case of large three-lane roundabout in Sheffield, UK *(35,37)*.

For each case estimates of capacity, degree of saturation (v/c ratio) and practical spare capacity from the aaSIDRA (version 2.1) and TRL (UK) linear regression models are compared. No calibration was carried out (i.e. all estimates are based on model defaults). Practical degrees of saturation are calculated using a target degree of saturation of 85 per cent.

A Single-lane Freeway Interchange Roundabout, Sydney, Australia

Queues on the freeway off-ramp at this single-lane small-size interchange roundabout in Sydney, Australia extended onto the freeway during evening peak hours (*Figure 12*). A large percentage of heavy vehicles use the freeway off-ramp. A queue of over 38 vehicles was observed for more than an hour.

Analysis is carried out for the 15-min pm peak period. The hourly flow rates calculated from 15-min peak volumes are shown in *Figure 12*. The geometry data are summarized in *Table 2*. Approach flaring is negligible at this roundabout.

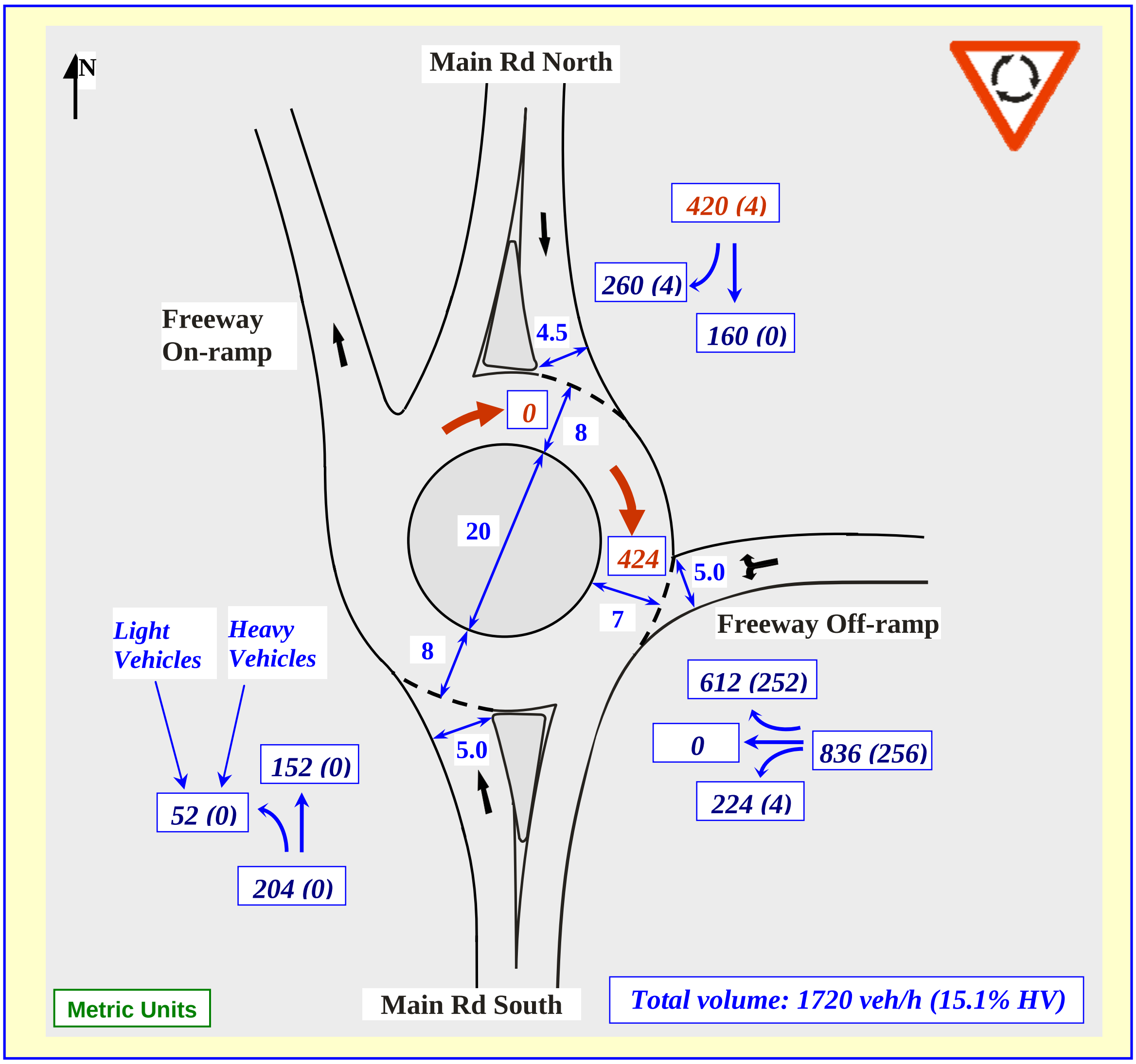

Figure 12 - An interchange roundabout case from Sydney, Australia.
Driving on the left-hand side of the road applies (see the Appendix for driving on the right-hand side of the road with US customary units).

Table 2 - Geometry data for the interchange roundabout in Sydney, Australia (Metric Units)

App. ID	Approach Name	Average entry lane width	Total entry width	App. half width	Flare length (effective)	Entry radius	Entry angle
		w_L (m)	w_e (m)	w_a (m)	L_f (m)	r_e (m)	Φ_e (deg)
S	Main Rd South	5.0	5.0	5.0	10	30	25
E	Freeway Off-ramp	5.0	5.0	5.0	10	18	20
N	Main Rd North	4.5	4.5	4.5	10	34	24
		Inscribed diameter	Central island diameter	Circulating road width	No of entry lanes	No of circulating lanes	
		D_i (m)	D_c (m)	w_c (m)	n_e	n_c	
S	Main Rd South	36	20	8.0	1	1	
E	Freeway Off-ramp	34	20	7.0	1	1	
N	Main Rd North	36	20	8.0	1	1	

Table 3 - Capacity results for the interchange roundabout in Sydney, Australia

App. ID	Approach Name	Total App. Flow (veh/h)	Circul. Flow **(1)** (pcu/h)	Total App. Capacity (veh/h)	Degree of saturation (v/c ratio)	Practical Spare Capacity (x_p = 0.85)
			aaSIDRA Model			
S	Main Rd South	204	**1128**	412	0.495	72%
E	Freeway Off-ramp	1092	424	**910**	**1.200**	**-29%**
N	Main Rd North	424	0	1751	0.242	251%
			TRL (UK) Linear Regression Model (2)			
S	Main Rd South	204	**1328**	561	0.364	134%
E	Freeway Off-ramp	1092	424	**1144**	**0.954**	**-11%**
N	Main Rd North	424	0	1575	0.269	216%

(1) The aaSIDRA circulating flow rate for the South approach includes capacity constraint effect due to oversaturation on East approach (x > 1.0). Circulating flows for the TRL (UK) model are without any capacity constraint since all approach lanes are estimated to be undersaturated (x < 1.0). All circulating flows include heavy vehicle effects (in pcu/h).

(2) The grade-separated roundabout option used for the TRL (UK) model.

Unbalanced flow conditions arise due to the origin-destination demand flow pattern at this roundabout. The circulating flow rate in front of the North approach is zero since no traffic can go to the freeway off-ramp! The flow from the approach can enter the roundabout continuously, and although this dominant flow rate is not high, it causes difficulty for entry from the freeway off-ramp. The high proportion of heavy vehicles in this stream contributes to the lower capacity on this approach.

Estimates of capacity, degree of saturation (v/c ratio) and practical spare capacity for the aaSIDRA and TRL (UK) linear regression models are given in *Table 3*. The entry capacities for the TRL (UK) linear regression model were calculated using the model for grade-separated roundabouts, and they have been adjusted for heavy vehicle effects using the aaSIDRA method. Circulating flow rates adjusted for heavy vehicle effects (pcu/h) were also used for both models.

In this case, aaSIDRA estimates oversaturated conditions for the freeway-off-ramp (x = 1.20). The queue length estimated by aaSIDRA for the freeway-off-ramp matches the observed values (average queue 31 vehicles, 95th percentile queue 95 vehicles). The TRL (UK) model estimates a large degree of saturation for the freeway-off-ramp (x = 0.95) although it is more optimistic than the aaSIDRA model. Using a sensitivity analysis in aaSIDRA, a degree of saturation of 0.95 was obtained to match the TRL (UK) model, in which case the aaSIDRA estimated low queue lengths not matching those observed (average queue 10 vehicles, 95th percentile queue 27 vehicles).

In aaSIDRA analysis, a "Medium" adjustment level was used for the effect of the ratio of approach flow to circulating flow for the North approach where the circulating flow rate is zero (default adjustment level in aaSIDRA 2.1). When a "Low" adjustment level is used, the capacity of the North approach drops to 1557 veh/h and the degree of saturation increases to 0.272, which is closer to the TRL (UK) model estimate. In this case, aaSIDRA estimates worse conditions on the freeway-off-ramp (x = 1.24, spare capacity = -32%, average queue 35 vehicles, 95th percentile queue 105 vehicles). This factor does not affect the results from the TRL (UK) model.

A Two-lane T-intersection Roundabout

This two-lane roundabout case (*Figure 13*) is based on an example presented by Chard (Case A) *(29,30)* who demonstrated the lack of sensitivity of the TRL (UK) linear regression model to different approach lane use arrangements. The case is presented for driving on the right-hand side of the road and with metric units. The volumes are modified in order to demonstrate the importance of unbalanced flow conditions as well as approach and circulating road lane use issues.

Chard's article addressed prediction problems associated with the "approach" method of traffic modeling which lumps traffic in individual lanes of an approach together irrespective of lane arrangements (exclusive or shared) and any unequal lane utilization (including the case of defacto exclusive lanes). Chard stated that "*(the TRL model) can take no account of either unused or unequally used lanes or flared sections on roundabout entry approaches. (The TRL model) is, in fact, completely 'blind' to such occurrences, and as a consequence may produce hopelessly optimistic predictions*."

Figure 13 shows a roundabout with two entry lanes and single-lane circulating road for all approach roads. Approach lane disciplines are as shown in Figure 5a of Chard. Irrespective of specifying a single-lane or two-lane circulating road, all circulating streams would operate effectively as single-lane movements due to exclusive lane arrangements on approach roads (this reduces the capacity of the roundabout). A variety of options are feasible for approach and circulating lane arrangements for this roundabout, using various combinations of approach roads with exclusive or shared lanes and single-lane or two-lane circulating roads.

Figure 14 shows an alternative arrangement with two-lane approach roads with shared lanes and two circulating lanes for all approach roads. This arrangement increases capacities due to the better balance of flows in approach lanes to make use of available lane capacities as well as better opportunity to accept gaps in multi-lane circulating streams.

Analysis is carried out for 15-min peak period. The hourly flow rates calculated from 15-min peak volumes are shown in *Figure 13*. The geometry data are used as specified by Chard as summarized in *Table 4*. The data given in *Table 4* are for the single-lane circulating road case as in *Figure 13*. For the two-lane circulating road case as in *Figure 14*, the circulating road width is 10 m and the central island diameter is 20 m. The inscribed diameter is D_i = 40 m in both cases.

The circulating flow in front of each approach consists of traffic from one approach only at this roundabout. The circulating flow rate in front of the East approach (Arm C) is high. This circulating stream enters from the South approach (Arm B). The circulating flow rate in front of the South approach is significant but not high. This indicates potential for unbalanced flow conditions.

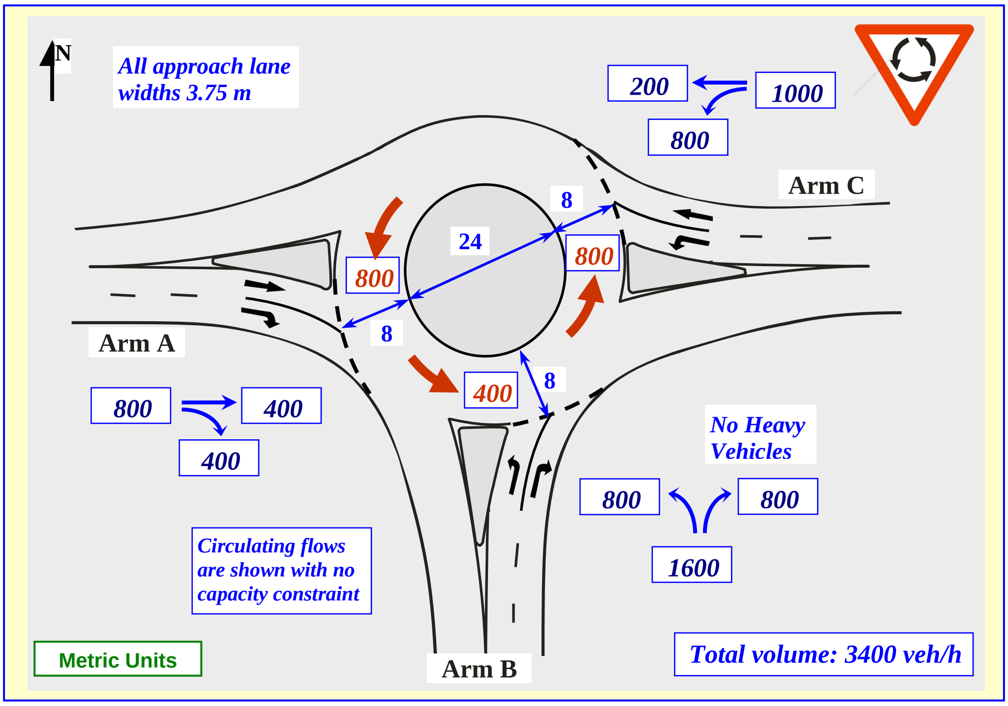

Figure 13 - A T-intersection roundabout case based on an example given by Chard *(29,30)*: exclusive approach lanes and single-lane circulating road

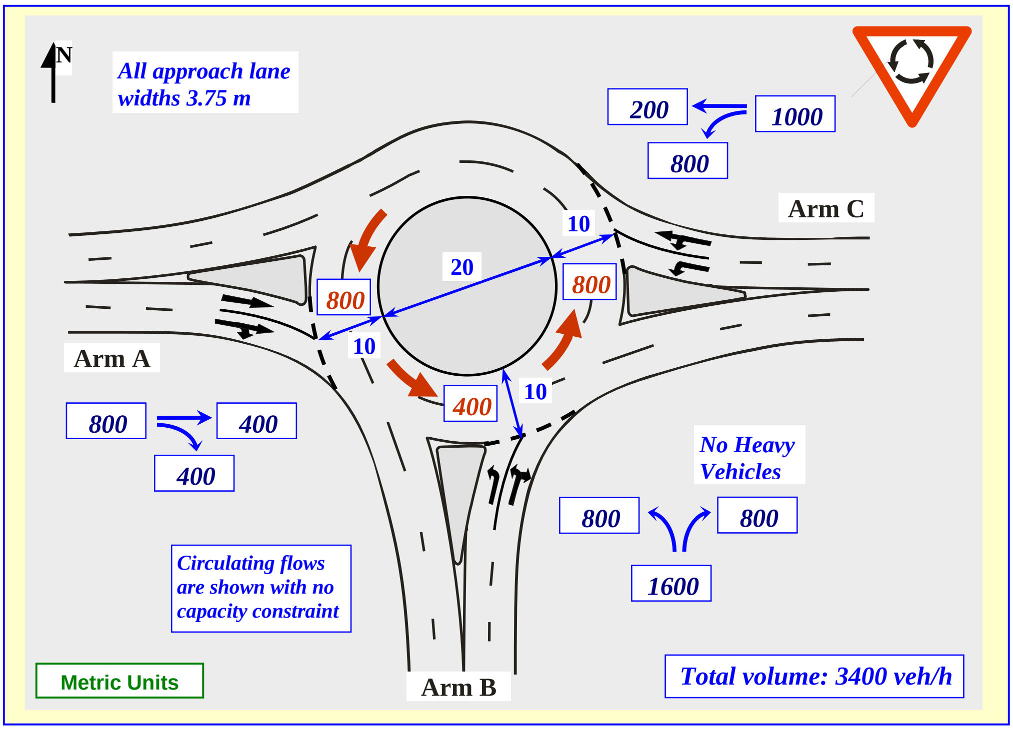

Figure 14 - A roundabout T-intersection case based on an example given by Chard *(29,30)*: shared approach lanes and two-lane circulating road

Table 4 - Geometry data for the T-intersection roundabout

Approach ID	Approach Name	Average entry lane width w_L (ft)	Total entry width w_e (ft)	App. half width w_a (ft)	Flare length (effective) L_f (ft)	Entry radius r_e (ft)	Entry angle Φ_e (deg)
W	Arm A	13 (3.75 m)	26 (7.50 m)	23 (6.0 m)	33 (10 m)	66 (20 m)	40
S	Arm B	13 (3.75 m)	26 (7.50 m)	23 (6.0 m)	33 (10 m)	66 (20 m)	40
E	Arm C	13 (3.75 m)	26 (7.50 m)	23 (6.0 m)	33 (10 m)	66 (20 m)	40
		Inscribed diameter D_i (ft)	Central island diameter D_c (ft)	Circulating road width w_c (ft)	No of entry lanes n_e	No of circulating lanes n_c	
W	Arm A	132 (40.0 m)	80 (24.0 m)	26.0 (8.0 m)	2	1 (2)	
S	Arm B	132 (40.0 m)	80 (24.0 m)	26.0 (8.0 m)	2	1 (2)	
E	Arm C	132 (40.0 m)	80 (24.0 m)	26.0 (8.0 m)	2	1 (2)	

For the ***single-lane*** circulating road case (n_c = 1) shown in *Figure 13*, the circulating road width is w_c = 26 ft (8 m) and the central island diameter is D_c = 80 ft (24 m). These are given above.

For the ***two-lane*** circulating road case (n_c = 2) shown in *Figure 14*, the circulating road width is w_c = 33 ft (10 m) and the central island diameter is D_c = 66 ft (20 m). The inscribed diameter is D_i = 132 ft (40 m in both cases).

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

Estimates of capacity, degree of saturation (v/c ratio) and practical spare capacity for the aaSIDRA and TRL (UK) linear regression models are given in *Table 5*. It is seen that aaSIDRA estimates differ significantly for the single-lane and two-lane circulating road cases whereas the TRL (UK) model estimates for the two cases are the same.

aaSIDRA estimates oversaturated conditions for the East approach (Arm C) in the case of single-lane circulating road with exclusive lanes (x = 1.09), but estimates satisfactory operating conditions in the case of two-lane circulating road with shared lanes (x = 0.71). aaSIDRA estimates more favorable gap-acceptance conditions in the case of two-lane circulating flows, and the approach lane use is more balanced with shared lanes. The TRL (UK) model estimates satisfactory conditions for both cases (x = 0.67). Assumptions of the "approach" method used in the TRL (UK) model are close to the case of two-lane circulating road with shared approach lanes, and therefore in close agreement with the aaSIDRA method.

Using a lane-by-lane method, aaSIDRA identifies critical lanes distinguishing between exclusive and shared lane cases and allowing for any unequal lane utilization. Combined with the unbalanced flow effects resulting from the O-D flow pattern and unfavorable gap-acceptance conditions presented by single-lane circulating flows, aaSIDRA is able to identify oversaturation on the East approach in the case of single-lane circulating road with exclusive lanes. On the other hand, the TRL capacity model combines exclusive and shared lanes to obtain an average approach degree of saturation, and therefore cannot identify unequal lane utilization and cannot distinguish between different lane use arrangements.

aaSIDRA estimates of delay, operating cost, fuel consumption and CO_2 emission comparing the case of single-lane circulating road with exclusive lanes vs the case of two-lane circulating road with shared lanes showed that, considering annual values of one hour of traffic operation only, the difference between the two cases amounted to approximately 9,000 person-hours of delay, US$72,000 in operating cost, 14,000 L of fuel consumption and 34,000 kg of CO_2 emission per year.

Table 5 - Capacity results for the T-intersection roundabout (see the Appendix for the results for driving on the right-hand side of the road with US customary units).

App. ID	Approach Name	Total App. Flow (veh/h)	Circul. Flow (1) (pcu/h)	Critical Lane (2)	Critical Lane Flow (veh/h)	Total App. Capacity (veh/h)	Critical Lane capacity (veh/h)	Degree of saturation (v/c ratio)	Practical Spare Capacity (x_p = 0.85)
			aaSIDRA: Single-lane circulating road and exclusive approach lanes						
W	Arm A	800	733	1 (T)	**400**	1435	629	0.635	34%
S	Arm B	1600	400	1 (L)	**800**	2167	984	0.813	5%
E	Arm C	1000	800	1 (L)	**800**	**1224**	**733**	**1.091**	**-22%**
			aaSIDRA: Two-lane circulating road and shared approach lanes						
W	Arm A	800	**800**	2 (TR)	**431**	1507	812	0.531	60%
S	Arm B	1600	400	2 (LR)	**841**	2050	1078	0.781	9%
E	Arm C	1000	800	2 (LT)	**537**	**1419**	**762**	**0.705**	**21%**
			TRL (UK) Linear Regression Model: same for both lane arrangements						
W	Arm A	800	**800**	-	-	1490	-	0.537	58%
S	Arm B	1600	400	-	-	1771	-	**0.904**	**-6%**
E	Arm C	1000	800	-	-	**1490**	-	**0.671**	**27%**

(1) The aaSIDRA circulating flow rate for the West approach includes capacity constraint effect due to oversaturation on East approach (x > 1.0) in the case of single-lane circulating road. Circulating flows for the TRL (UK) model, as well as the aaSIDRA model for two-lane circulating road, are without any capacity constraint since all approach lanes are estimated to be undersaturated (x < 1.0).

(2) aaSIDRA approach degrees of saturation represent the critical lane degrees of saturation (L: Left, T: Through, R: Right). The TRL capacity model combines exclusive and shared lanes to obtain an average approach degree of saturation, and therefore cannot identify unequal lane utilization and cannot distinguish between different lane use arrangements.

Moore Street Roundabout, Sheffield, UK

Analyses of traffic conditions at Moore Street roundabout in Sheffield, United Kingdom were published in UK papers *(35,37)*. This large three-lane roundabout is shown in *Figure 15*. The analysis is presented for the morning peak period when capacity problems were observed. The geometry data are summarized in *Table 6*.

This roundabout presents an interesting case of unbalanced flows and varying lane use arrangements on roundabout approaches. The effect of exclusive lanes that require consideration of lane groups is also an interesting aspect of this case.

A very heavy traffic stream enters from St Mary's Gate with over 1700 veh/h turning right or making a U turn. The next two entries (Ecclesall Road and Hanover Way) "suffer great difficulties" and the intersection is "over capacity" *(35)*. Part-time signals were introduced in order to "control the dominant flow from St Mary's Gate and to provide more gaps in the circulatory stream for entering vehicles in the next two entries" *(37)*. The analysis presented in this paper is for the conditions before the introduction of metering signals.

The analysis is carried out for 15-min peak conditions using a Peak Flow Factor of 0.92 (volumes are approximately 9 per cent higher than those shown in *Figure 15*. The volumes shown in *Figure 15* are based on the values listed in Shawaly, et al, Table I *(37)* for "Before (introduction of signals)" and "Morning Peak" conditions, and include adjustment for any heavy vehicles using a factor of 2 pcu/veh.

Table 6 - Geometry data for Moore Street roundabout, Sheffield, UK (Metric Units)

App. ID	Approach Name	Average entry lane width	Total entry width	App. half width	Flare length (effective)	Entry radius	Entry angle
		w_L (m)	w_e (m)	w_a (m)	L_f (m)	r_e (m)	Φ_e (deg)
SE	St Mary's Gate	4.67	14.0	11.0	45	95	24
NE	Moore St	3.83	11.5	7.3	45	35	30
NW	Hanover Way	3.33	10.0	7.3	45	65	35
SW	Ecclesall Rd	4.73	14.2	10.6	45	45	39
		Inscribed diameter	Central island diameter	Circulating road width	No of entry lanes	No of circulating lanes	
		D_i (m)	D_c (m)	w_c (m)	n_e	n_c	
SE	St Mary's Gate	76	46	15.0	3	3	
NE	Moore St	81	51	15.0	3	2	
NW	Hanover Way	76	46	15.0	3	3	
SW	Ecclesall Rd	81	51	15.0	3	3	

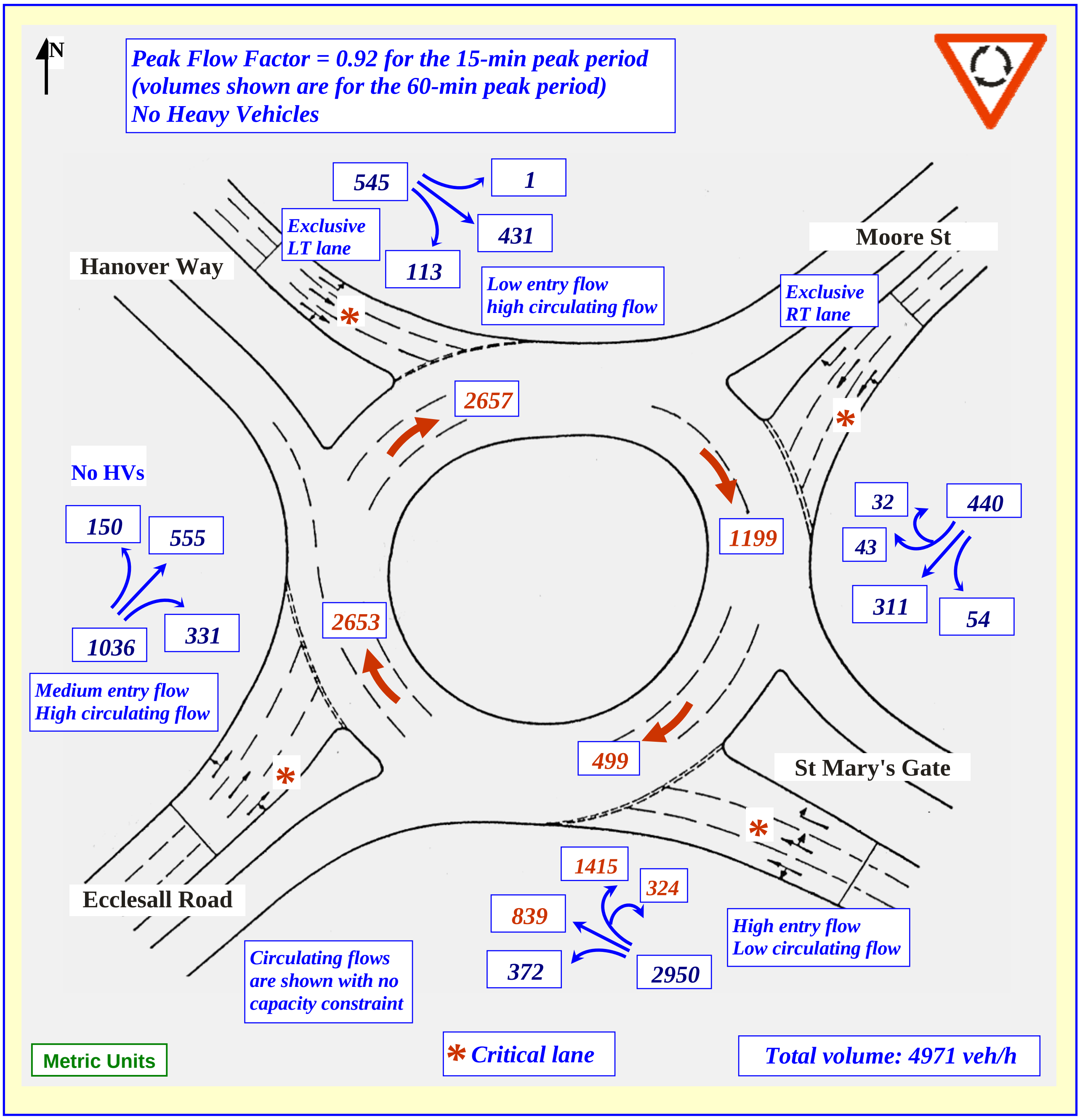

Figure 15 - Moore Street roundabout, Sheffield, UK (35):
traffic volumes for the morning peak hour period shown
Driving on the left-hand side of the road applies ***(see the Appendix for driving on the right-hand side of the road with US customary units).***

Estimates of capacity, degree of saturation (v/c ratio) and practical spare capacity for the aaSIDRA and TRL (UK) linear regression models are given in *Table 7*. While there is good agreement between the aaSIDRA and the TRL (UK) linear regression models for the conditions on St Mary's Gate approach, significant differences are observed for other approaches:

(i) For Ecclesall Road, aaSIDRA indicates oversaturated conditions with a high degree of saturation (1.37) and negative spare capacity (-38%) whereas the TRL (UK) linear regression model indicates good operating conditions, i.e. a low degree of saturation (0.66) and large spare capacity (29%). aaSIDRA identifies unequal lane utilization on this approach, indicating a defacto exclusive right-turn lane, which is the critical lane.

(ii) For Hanover Way, aaSIDRA indicates a fairly high degree of saturation (0.82) compared with a low degree of saturation (0.59) estimated by the TRL (UK) model. Exclusive left-turn lane on this approach carries negligible traffic and the aaSIDRA method discounts the capacity of this lane. The TRL (UK) method gives a low degree of saturation since it does not account for this unequal use of lanes on the approach.

(iii) For Moore St, the degrees of saturation estimated by both models are similar. Although there is unequal lane utilization due to the exclusive right-turn lane on this approach, the TRL (UK) model estimates a lower approach capacity which compensates for inability to recognize unequal lane use.

Table 7 - Capacity results for Moore Street Roundabout, Sheffield, UK
(Morning 15-min Peak Period)

App. ID	Approach Name	Total App. Flow (veh/h)	Circul. Flow (1) (pcu/h)	Critical Lane (2)	Critical Lane Flow (veh/h)	Total App. Capacity (veh/h)	Critical Lane capacity (veh/h)	Degree of saturation (v/c ratio)	Practical Spare Capacity (x_p = 0.85)
			aaSIDRA Model						
SE	St Mary's Gate	3206	542	2 (TR)	**1223**	3755	1433	0.854	0%
NE	Moore St	479	**1207**	1 (LT)	**240**	**3205**	**1286**	0.187	355%
NW	Hanover Way	592	**2716**	2 (T)	**357**	1152	**434**	**0.823**	**3%**
SW	Ecclesall Rd	1126	2884	3 (R)	**360 (3)**	**934**	**263**	**1.367**	**-38%**
			TRL (UK) Linear Regression Model						
SE	St Mary's Gate	3206	542	-	-	3842	-	0.834	2%
NE	Moore St	479	**1303**	-	-	**2343**	-	0.204	325%
NW	Hanover Way	592	**2888**	-	-	999	-	**0.592**	**44%**
SW	Ecclesall Rd	1126	2884	-	-	**1714**	-	**0.657**	**29%**

(1) Circulating flows for TRL model are without any capacity constraint since all approach lanes are estimated to be undersaturated (x < 1.0). aaSIDRA circulating flows are subject to capacity constraint.

(2) aaSIDRA approach degree of saturation represents the critical lane degree of saturation (L: Left, T: Through, R: Right). The TRL capacity model combines exclusive and shared lanes on Moore St and Hanover Way to obtain an average approach degree of saturation.

(3) aaSIDRA identifies defacto exclusive right-turn lane on Ecclesall Rd.

A lane-by-lane analysis method as used by aaSIDRA *(12)*, or a lane group method as used in the US Highway Capacity Manual *(1),* is needed to identify unequal lane use, including a defacto exclusive lane case, identified by aaSIDRA for Ecclesall Road, Hanover Way and Moore St approaches.

A more detailed discussion is presented below for the Ecclesall Rd entry. For this approach, aaSIDRA indicates oversaturated conditions. The origin - destination (O-D) effect in the aaSIDRA model is determined considering the components of the circulating flow. For the Ecclesall Rd entry, the total circulating flow rate is 2884 pcu/h for the 15-min peak period. This includes 2802 pcu/h (97 per cent) from the dominant approach (St Mary's Gate) with 79 per cent queued traffic, and negligible traffic from Moore St. Therefore, this heavy traffic from St Mary's Gate approach acts as a dominant flow which reduces the capacity of the Ecclesall Rd approach significantly.

The paper by Shawaly, et al, Figure 3-5 *(37)* indicates observed capacity values in the range 520 to 1840 veh/h with an average value of around 1200 veh/h for the circulating flow rate of around 2650 pcu/h. This is close to the aaSIDRA estimate for the 60-min peak period (1244 veh/h), and much lower than the TRL (UK) method estimate (1903 veh/h). For the 15-min peak period, aaSIDRA estimates a total approach capacity of 934 whereas the TRL (UK) method estimates 1714 veh/h).

Thus, the aaSIDRA model is seen to reflect the capacity problems at this high-demand, unbalanced flow roundabout. On the other hand, the TRL (UK) linear regression estimates good operating conditions for all approaches. This finding is in line with other case studies described in this paper.

CONCLUDING REMARKS

The case studies of roundabouts from Australia, UK and USA described in this paper highlight systematic differences between the aaSIDRA and TRL (UK) linear regression models and explain possible reasons for the contradictory results that may be obtained from these models. Such systematic model differences have important practical design implications. The roundabouts presented in this paper display unbalanced demand flow patterns (with dominant flows that impose priority emphasis) as well as unequal approach lane use and different circulating lane use cases. These are important factors contributing to significant differences between the aaSIDRA and other models. The old Australian (NAASRA) model that uses fixed gap-acceptance parameters *(5)* and the method given in the current Australian Roundabout Guide *(8)* also fail to account for these factors fully. The method described in the FHWA Roundabout Guide *(52)* is expected to give similar results to the TRL (UK) model it is based on.

Comments on model differences and possible reasons for the TRL (UK) linear regression model to give lower capacities at low circulating flows and higher capacities at high circulating flows are given in a recent publication *(22)*.

This paper focused on comparison of two widely used analytical models. Microsimulation models offer a great potential for modeling complex gap-acceptance situations experienced in many situations in urban traffic. Modeling issues discussed in this paper are also applicable to microsimulation models since driver behavior rules and gap-acceptance parameter values used in microsimulation will affect the resulting capacity and performance estimates *(31)*. Comparisons of capacity and performance estimates from different microsimulation models and between microsimulation and analytical models are also recommended.

ACKNOWLEDGEMENTS

The author is the developer of the aaSIDRA model, and comments presented in this paper regarding other models should be read with this in mind. The comments about the TRL (UK) linear regression model are relevant to the original published model and are valid for software packages using that model only to the extent that the original model is used without modification to address the issues raised in this paper.

AUTHOR INFORMATION

Rahmi Akçelik
Adjunct Professor, Monash University,
Director, Akcelik and Associates Pty Ltd
P.O. Box 1075G, Greythorn Victoria 3104, Australia
Phone: +613 9857 9351
Fax: +613 9857 5397
Email: rahmi@akcelik.com.au
Website: www.aatraffic.com.au

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Appendix - Case Studies
(Right-Hand versions and additional information)

Case studies are presented to show that roundabout capacity and level of service depend not only on the circulating flow rate but also the characteristics of approach flows contributing to the circulating flow:

- **the amount of queuing on the approach road**
- **circulating lane use**

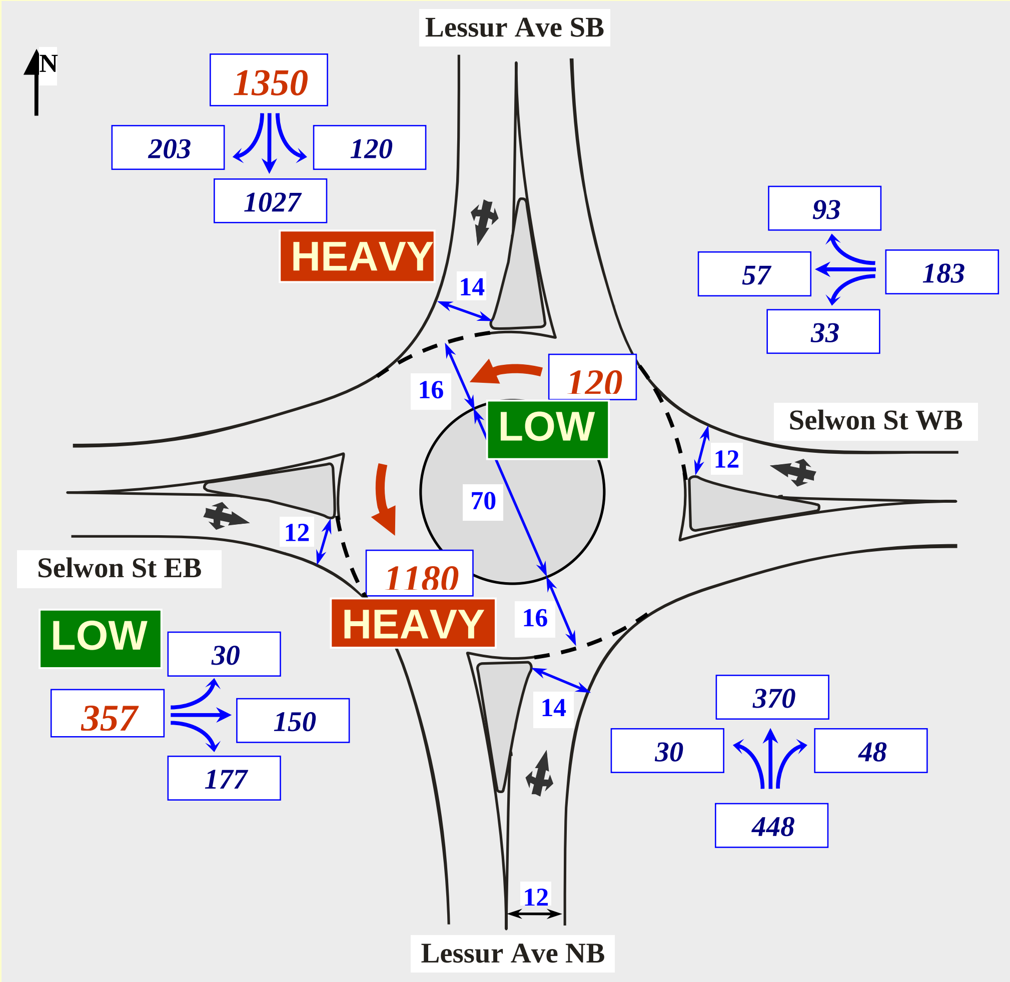
Country
- **Australia**
- **UK**
- **USA**

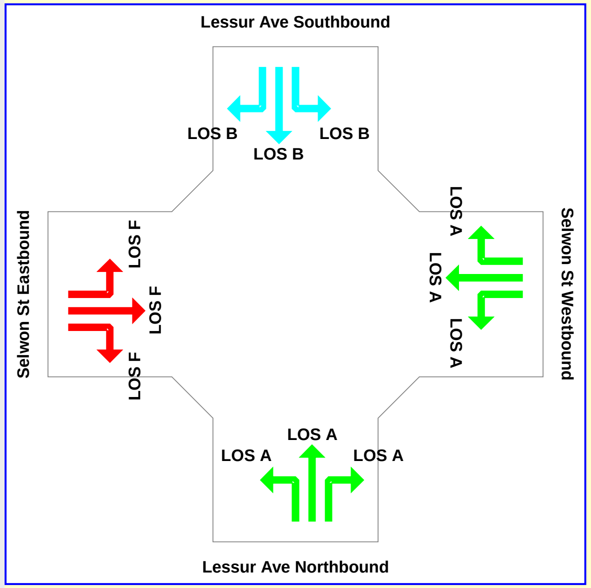
Roundabout size
- **1-lane**
- **2-lane**
- **3-lane**

Demand volumes
- **Large**
- **Small**

1700 to 5300 veh/h

A Single-Lane Roundabout Case, USA





Peaking parameters:
$T = 60$ min, $T_p = 15$ min
PFF = 1.00
No Heavy Vehicles

US Customary Units

Roundabout geometry data for the single-lane US roundabout case

Approach ID	Approach Name	Average entry lane width w_L (ft)	Total entry width w_e (ft)	App. half width w_a (ft)	Flare length (effective) L_f (ft)	Entry radius r_e (ft)	Entry angle Φ_e (deg)
W	Selwon St EB	12 (3.66 m)	12 (3.66 m)	10 (3.16 m)	66 (20 m)	100 (30.5 m)	30
S	Lessur Ave NB	14 (4.27 m)	14 (4.27 m)	12 (3.77 m)	66 (20 m)	70 (21.3 m)	30
E	Selwon St WB	12 (3.66 m)	12 (3.66 m)	10 (3.16 m)	66 (20 m)	120 (36.6 m)	30
N	Lessur Ave SB	14 (4.27 m)	14 (4.27 m)	10 (3.77 m)	66 (20 m)	80 (24.4 m)	30
		Inscribed diameter D_i (ft)	Central island diameter D_c (ft)	Circulating road width w_c (ft)	No of entry lanes n_e	No of circulating lanes n_c	
W	Selwon St EB	102 (31.1 m)	70 (21.3 m)	16.0 (4.9 m)	1	1	
S	Lessur Ave NB	102 (31.1 m)	70 (21.3 m)	16.0 (4.9 m)	1	1	
E	Selwon St WB	102 (31.1 m)	70 (21.3 m)	16.0 (4.9 m)	1	1	
N	Lessur Ave SB	102 (31.1 m)	70 (21.3 m)	16.0 (4.9 m)	1	1	

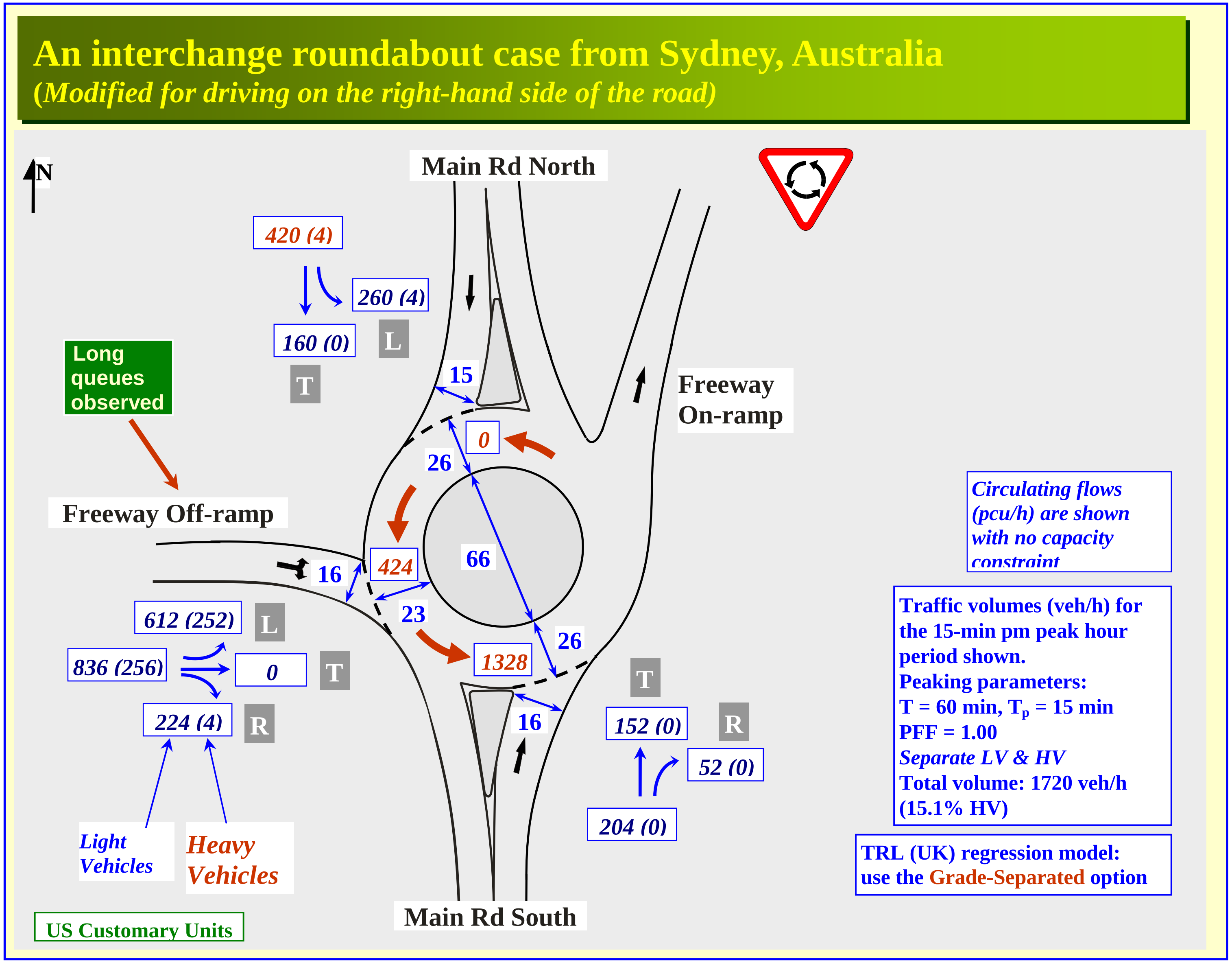
Data in metric units are shown in brackets.

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

Capacity estimates from various models for the single-lane US roundabout case

App. ID	Approach Name	Approach flow rate (veh/h)	aaSIDRA			TRL (UK) Linear Regression		
			Capacity (veh/h)	Degree of saturation (v/c ratio)	Practical Spare Capacity (x_p = 0.85)	Capacity (veh/h)	Degree of saturation (v/c ratio)	Practical Spare Capacity (x_p = 0.85)
W	**Selwon St EB**	**357**	**269**	**1.325**	**-36%**	**533**	**0.669**	**27%**
S	Lessur Ave NB	448	1112	0.403	111%	1121	0.400	113%
E	Selwon St WB	183	906	0.202	321%	888	0.206	313%
N	**Lessur Ave SB**	**1350**	**1389**	**0.972**	**-13%**	**1225**	**1.102**	**-23%**
App. ID	Approach Name	Approach flow rate (veh/h)	**NAASRA 1986**			**HCM 2000 Average**		
			Capacity (veh/h)	Degree of saturation (v/c ratio)	Practical Spare Capacity (x_p = 0.85)	Capacity (veh/h)	Degree of saturation (v/c ratio)	Practical Spare Capacity (x_p = 0.85)
W	**Selwon St EB**	**357**	**439**	**0.813**	**5%**	**544**	**0.656**	**30%**
S	Lessur Ave NB	448	1378	0.325	161%	1010	0.444	92%
E	Selwon St WB	183	1213	0.151	463%	895	0.205	315%
N	**Lessur Ave SB**	**1350**	**1625**	**0.831**	**2%**	**1155**	**1.169**	**-27%**

Capacity constraint applies to all models (different circulating flows used as a result)


An interchange roundabout case from Sydney, Australia
(Modified for driving on the right-hand side of the road)
N
Main Rd North
420 (4)
260 (4)
160 (0)
L
T
Long queues observed
15
0
26
Freeway On-ramp
Freeway Off-ramp
16
424
66
23
26
1328
612 (252)
L
836 (256)
0
T
224 (4)
R
T
16
152 (0)
R
52 (0)
204 (0)
Light Vehicles
Heavy Vehicles
Main Rd South
US Customary Units
Circulating flows (pcu/h) are shown with no capacity constraint
Traffic volumes (veh/h) for the 15-min pm peak hour period shown.
Peaking parameters:
T = 60 min, Tp = 15 min
PFF = 1.00
Separate LV & HV
Total volume: 1720 veh/h
(15.1% HV)
TRL (UK) regression model: use the Grade-Separated option

Geometry data for the interchange roundabout in Sydney, Australia
(For driving on the right-hand side of the road)

Approach ID	Approach Name	Average entry lane width w_L (ft)	Total entry width w_e (ft)	App. half width w_a (ft)	Flare length (effective) L_f (ft)	Entry radius r_e (ft)	Entry angle Φ_e (deg)
S	Main Rd South	16 (5.0 m)	16 (5.0 m)	16 (5.0 m)	33 (10 m)	100 (30 m)	25
W	Freeway Off-ramp	16 (5.0 m)	16 (5.0 m)	16 (5.0 m)	33 (10 m)	60 (18 m)	20
N	Main Rd North	15 (4.5 m)	15 (4.5 m)	15 (4.5 m)	33 (10 m)	110 (34 m)	24
		Inscribed diameter D_i (ft)	Central island diameter D_c (ft)	Circulating road width w_c (ft)	No of entry lanes n_e	No of circulating lanes n_c	
S	Main Rd South	118 (36.0 m)	66 (20.0 m)	26.0 (8.0 m)	1	1	
W	Freeway Off-ramp	112 (34.0 m)	66 (20.0 m)	23.0 (7.0 m)	1	1	
N	Main Rd North	118 (36.0 m)	66 (20.0 m)	26.0 (8.0 m)	1	1	

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

Capacity results for the interchange roundabout in Sydney, Australia
(For driving on the right-hand side of the road using US customary units)

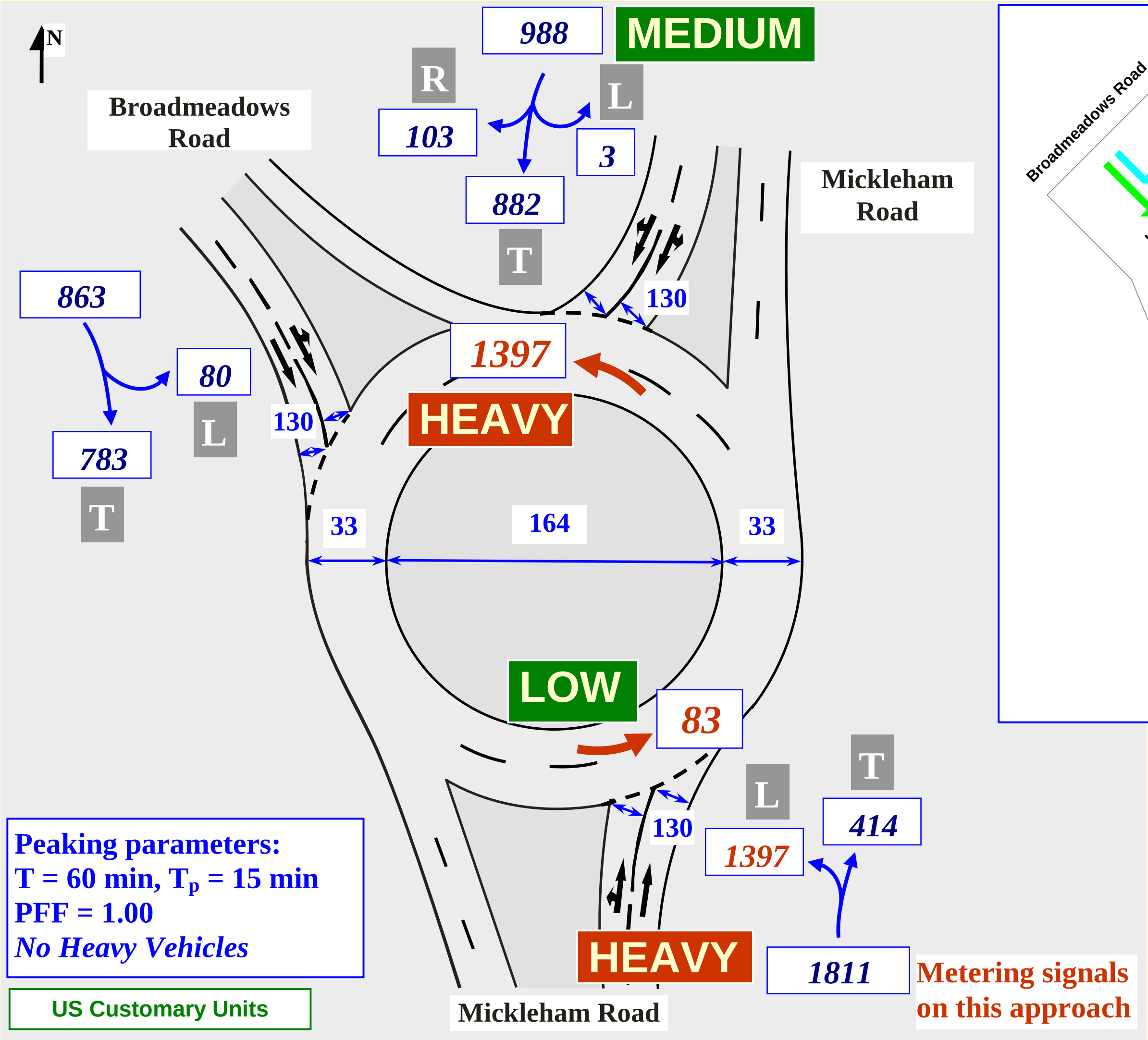
App. ID	Approach Name	Total App. Flow (veh/h)	Circul. Flow (1) (pcu/h)	Total App. Capacity (veh/h)	Degree of saturation (v/c ratio)	Practical Spare Capacity (x_p = 0.85)
			aaSIDRA Model			
S	Main Rd South	204	**1135**	416	0.491	73%
W	Freeway Off-ramp	1092	424	**896**	**1.219**	**-30%**
N	Main Rd North	424	0	1750	0.242	251%
			TRL (UK) Linear Regression Model (2)			
S	Main Rd South	204	**1328**	534	0.382	123%
W	Freeway Off-ramp	1092	424	**1099**	**0.994**	**-14%**
N	Main Rd North	424	0	1621	0.262	224%

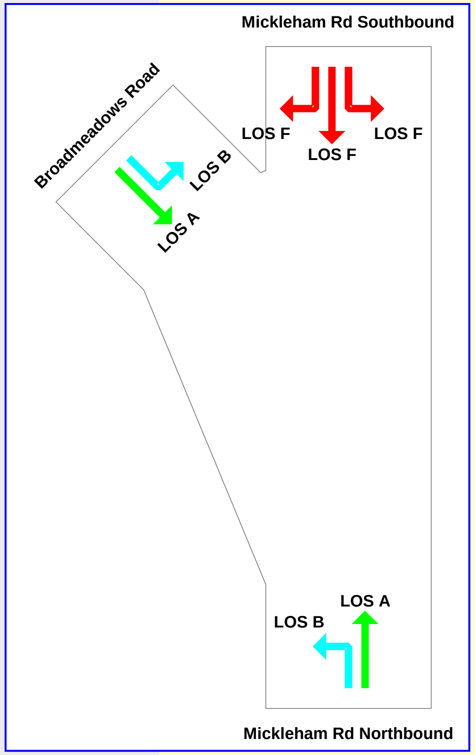
(1) The aaSIDRA circulating flow rate for the South approach includes capacity constraint effect due to oversaturation on West approach (x > 1.0). Circulating flows for the TRL (UK) model are without any capacity constraint since all approach lanes are estimated to be undersaturated (x < 1.0). All circulating flows include heavy vehicle effects (in pcu/h).

(2) The grade-separated roundabout option used for the TRL (UK) model.

Mickleham Rd and Broadmeadows Rd Roundabout, Melbourne, Australia

(Modified for driving on the right-hand side of the road)





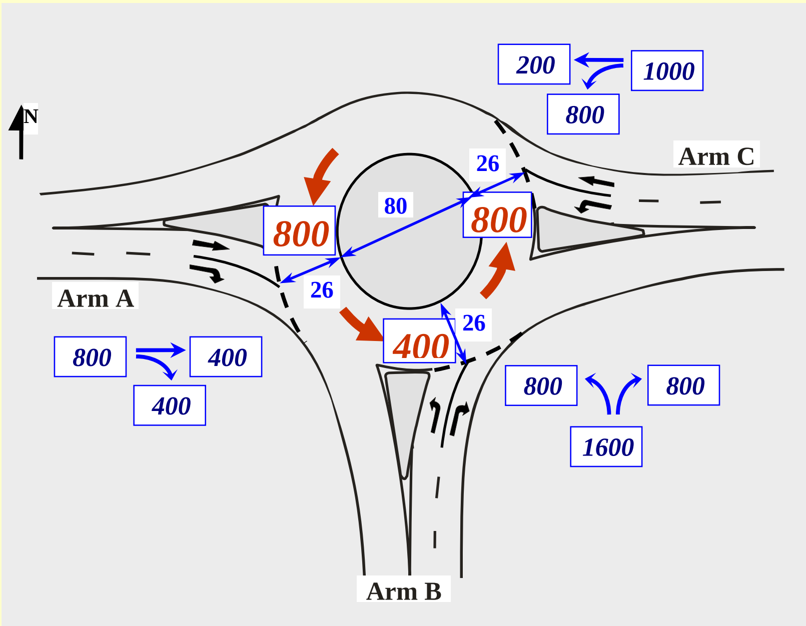
Effect of lane disciplines

- Lane disciplines for South approach: **L, LT**
- Broadmeadows Rd exit 2 lanes

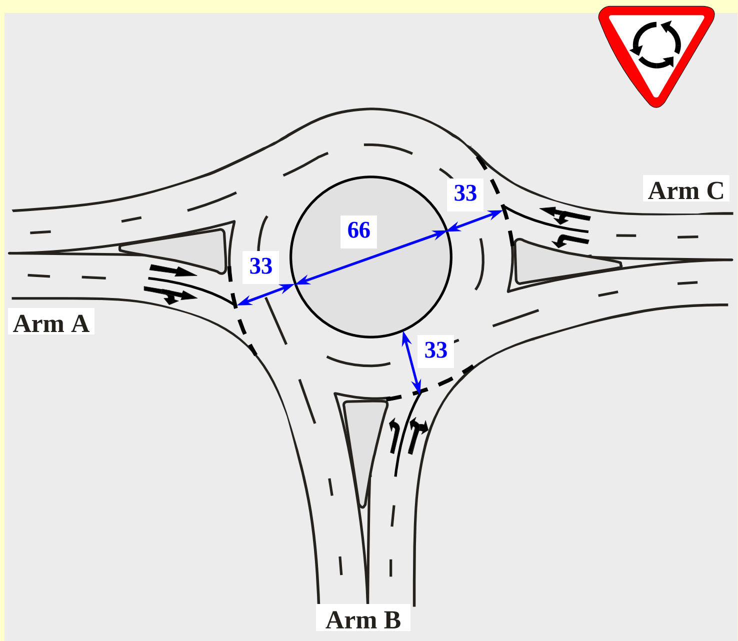
T-intersection roundabout case (based on article by CHARD, UK)

(Modified for driving on the right-hand side of the road)

Exclusive approach lanes and single-lane circulating road



Shared approach lanes and two-lane circulating road



All approach lane widths 13 ft

Circulating flows are shown with no capacity constraint

Peaking parameters:
T = 60 min, T_p = 15 min
PFF = 1.00
No Heavy Vehicles

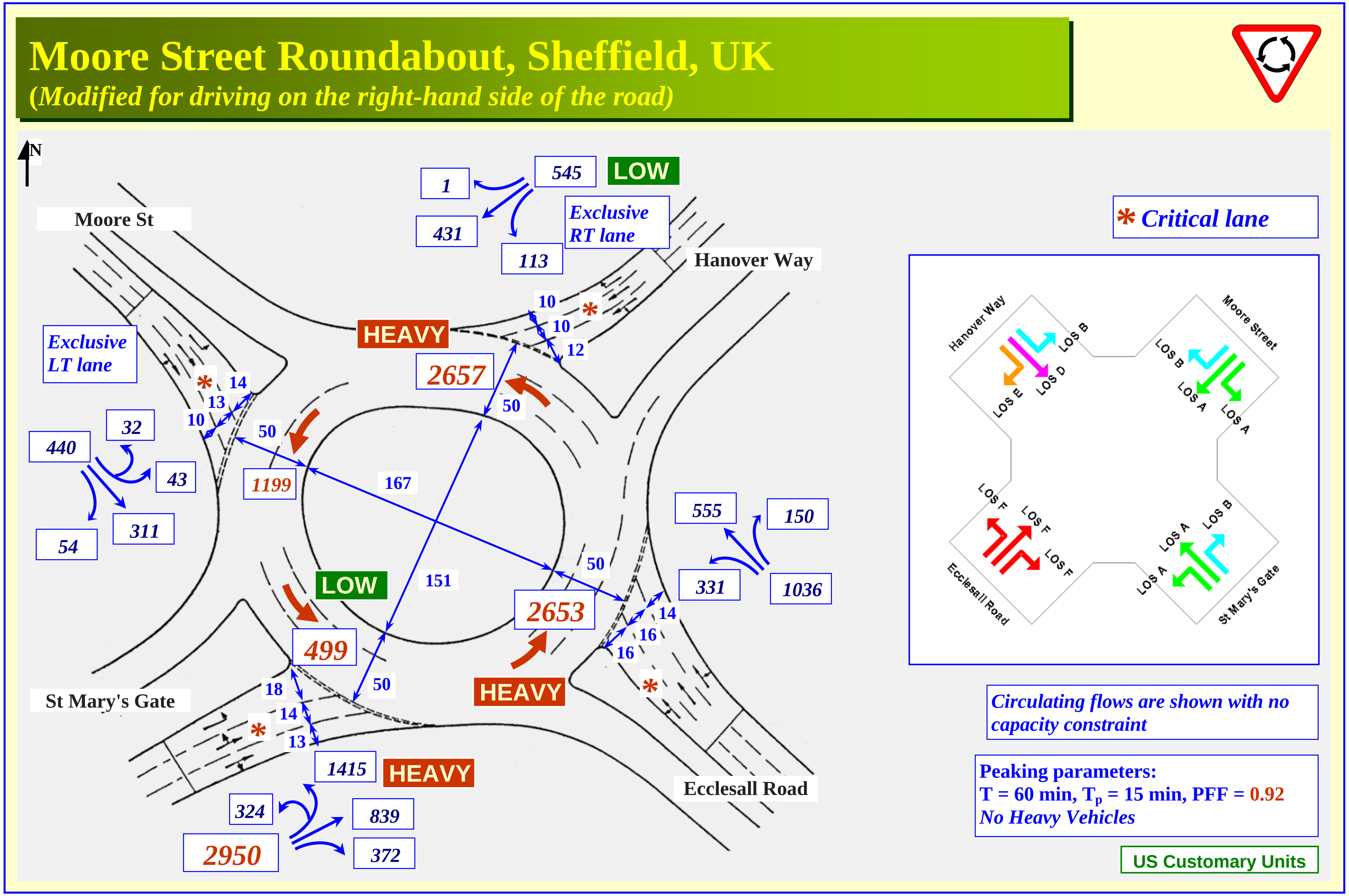
US Customary Units

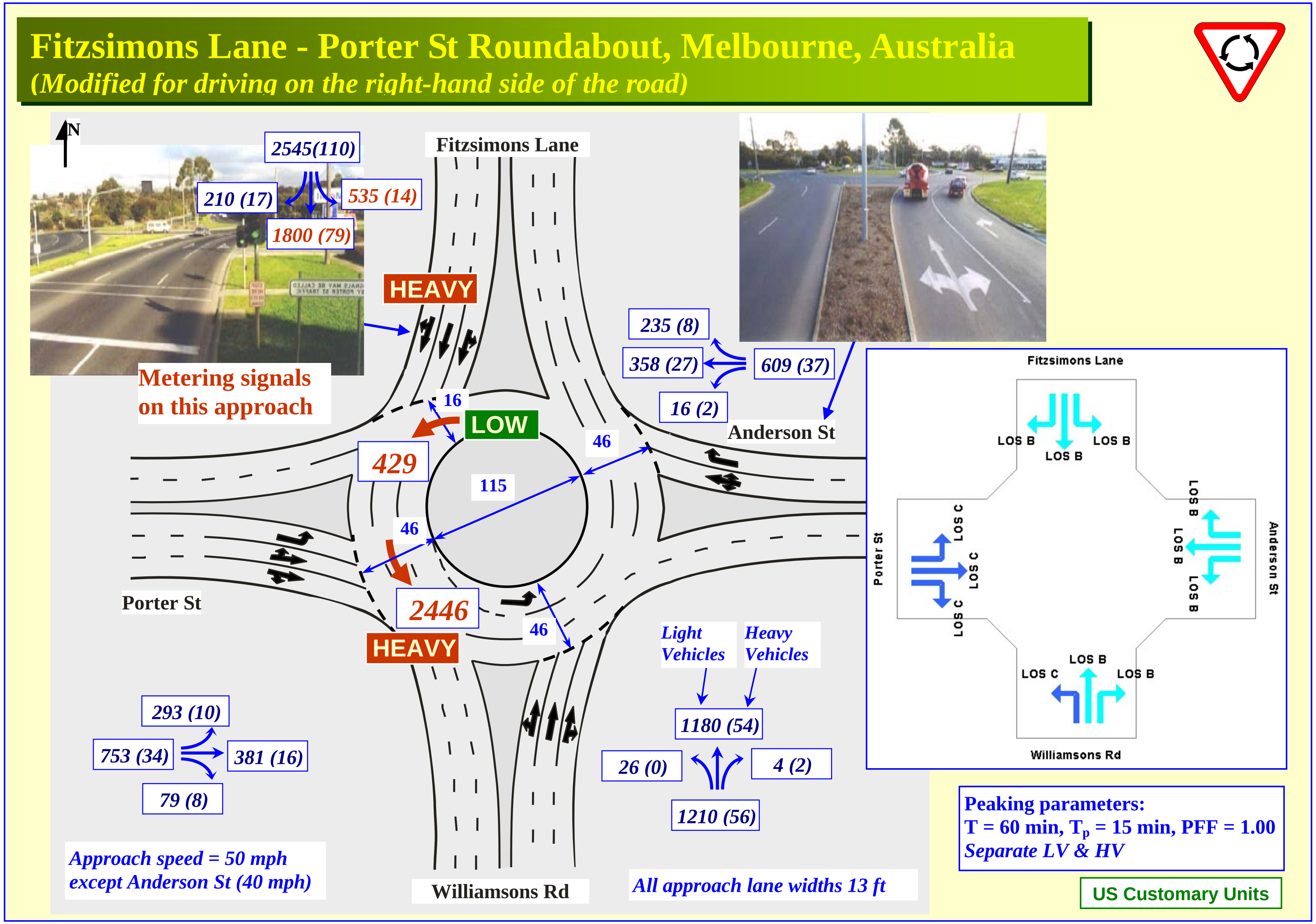
Capacity results for the T-intersection roundabout
(For driving on the right-hand side of the road using US customary units)

App. ID	Approach Name	Total App. Flow (veh/h)	Circul. Flow **(1)** (pcu/h)	Critical Lane **(2)**	Critical Lane Flow (veh/h)	Total App. Capacity (veh/h)	Critical Lane capacity (veh/h)	Degree of saturation (v/c ratio)	Practical Spare Capacity (x_p = 0.85)
			aaSIDRA: Single-lane circulating road and exclusive approach lanes						
W	Arm A	800	**749**	1 (T)	**400**	1433	628	0.637	33%
S	Arm B	1600	400	1 (L)	**800**	2199	999	0.801	6%
E	Arm C	1000	800	1 (L)	**800**	**1255**	**749**	**1.068**	**-20%**
			aaSIDRA: Two-lane circulating road and shared approach lanes						
W	Arm A	800	**800**	2 (TR)	**430**	1569	842	0.511	66%
S	Arm B	1600	400	2 (LR)	**841**	2091	1099	0.765	11%
E	Arm C	1000	800	2 (LT)	**537**	**1471**	**789**	**0.680**	**25%**
			TRL (UK) Linear Regression Model: Same results for both lane arrangements						
W	Arm A	800	**800**	-	-	1654	-	0.484	76%
S	Arm B	1600	400	-	-	1950	-	0.820	4%
E	Arm C	1000	800	-	-	**1654**	-	**0.605**	**41%**

(1) The aaSIDRA circulating flow rate for the West approach includes capacity constraint effect due to oversaturation on East approach ($x > 1.0$) in the case of single-lane circulating road. Circulating flows for the TRL (UK) model, as well as the aaSIDRA model for two-lane circulating road, are without any capacity constraint since all approach lanes are estimated to be undersaturated ($x < 1.0$).

(2) aaSIDRA approach degrees of saturation represent the critical lane degrees of saturation (L: Left, T: Through, R: Right). The TRL capacity model combines exclusive and shared lanes to obtain an average approach degree of saturation, and therefore cannot identify unequal lane utilization and cannot distinguish between different lane use arrangements.


Moore Street Roundabout, Sheffield, UK
(Modified for driving on the right-hand side of the road)
N
Moore St
Hanover Way
St Mary's Gate
Ecclesall Road
Exclusive RT lane
Exclusive LT lane
LOW
HEAVY
2657
1199
499
2653
2950
1
545
431
113
32
440
43
311
54
555
150
331
1036
1415
324
839
372
10
10
12
14
13
10
50
50
167
151
50
50
14
16
16
18
14
13
Critical lane
Hanover Way
Moore Street
Ecclesall Road
St Mary's Gate
LOS A
LOS B
LOS D
LOS E
LOS F
Circulating flows are shown with no capacity constraint
Peaking parameters:
T = 60 min, Tp = 15 min, PFF = 0.92
No Heavy Vehicles
US Customary Units


Fitzsimons Lane - Porter St Roundabout, Melbourne, Australia
(Modified for driving on the right-hand side of the road)
N
Fitzsimons Lane
2545(110)
210 (17)
535 (14)
1800 (79)
HEAVY
Metering signals on this approach
16
LOW
429
115
46
46
46
2446
HEAVY
235 (8)
358 (27)
609 (37)
16 (2)
Anderson St
Porter St
Light Vehicles
Heavy Vehicles
1180 (54)
26 (0)
4 (2)
1210 (56)
293 (10)
753 (34)
381 (16)
79 (8)
Approach speed = 50 mph except Anderson St (40 mph)
Williamsons Rd
All approach lane widths 13 ft
Fitzsimons Lane
LOS B
LOS B
LOS B
Porter St
LOS C
LOS C
LOS C
Anderson St
LOS B
LOS B
LOS B
LOS B
LOS C
LOS B
Williamsons Rd
Peaking parameters:
T = 60 min, Tp = 15 min, PFF = 1.00
Separate LV & HV
US Customary Units